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# Background Document on Proposed Liner and Leak Detection Rule

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The purpose of this document is to provide the technical rationale and support for the 3 main portions of the proposed Liner Leak Detection Rule: (1) leak detection system requirements; (2) extension of the double-liner system requirements to waste piles, significant unused portions and certain other units; and (3) construction quality assurance program requirements.

The portion of this document devoted to leak detection (Chapter 2) will: (1) present the technologies and materials available to construct lining systems and leak detection systems to meet the proposed regulations; (2) recommend best demonstrated available technologies (BDAT) to meet the requirements stemming from these regulations; and (3) quantify leak detection system performance capabilities associated with BDAT.

This document, to the greatest extent possible, presents a state-of-the-art review of available technologies and achievable performance levels in leak detection systems at hazardous waste management units. This review is necessary to establish the best demonstrated available technology (BDAT) for leak detection systems. It provides the critical information considered in developing EPA's regulatory options and describes the criteria used to select among the options. The document also presents a rationale and technical data to support extension of the double-liner system requirements to waste piles, significant unused portions, and certain other units. Lastly, the document presents a comprehensive discussion of the issues, methodologies, and benefits associated with construction quality assurance programs at hazardous waste management facilities.

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# CHAPTER 5
# LAND TREATMENT UNITS

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# CHAPTER 1

# INTRODUCTION

## 1.1 LEGISLATIVE HISTORY

### 1.1.1 Solid Waste Disposal Act/Resource Conservation and Recovery Act

The Solid Waste Disposal Act was substantively amended by the Resource Conservation and Recovery Act (RCRA) of 1976 which, with protection of human health and the environment as its objective, instituted a "cradle to the grave" management system to ensure the safe treatment, storage and disposal of hazardous waste.

Statutory changes made by the Resource Conservation and Recovery Act were so comprehensive that references to the statute dealing with hazardous waste management are made to the Resource Conservation and Recovery Act, although technically it is codified as the Solid Waste Disposal Act. This document will follow the tradition of citing the Resource Conservation and Recovery Act.

### 1.1.2 Hazardous and Solid Waste Amendments of 1984

#### 1.1.2.1 Hazardous and Solid Waste Amendment Policies and Findings

The Hazardous and Solid Waste Amendments of 1984 (HSWA) made many changes in Resource Conservation and Recovery Act sections covering regulation of hazardous waste.

Congressional findings relating to the threat to human health and the environment were the rationale for mandating hazardous waste management regulatory changes. Two of these findings acknowledged that "placement of inadequate controls on hazardous waste management will result in substantial risks to human health and the environment" (RCRA 1002(b)(5) as amended) and that "if hazardous waste management is improperly performed in the first instance, corrective action is likely to be expensive, complex, and time consuming" (RCRA 1002(b)(6)).

The HSWA also established a national policy that (hazardous) waste "should be treated, stored or disposed so as to minimize the present and future threat to human health and the environment" (RCRA 1003(b)).

The minimum technology requirements of the HSWA require the Environmental Protection Agency (EPA) to revise regulations for liners and leak detection systems at hazardous waste management units. The EPA is preparing to address through regulation three objectives mandated by the HSWA:

- To revise existing minimum technology requirements for landfills, surface impoundments, and waste piles;
- To establish minimum standards for leak detection and response; and,
- To augment existing rules with construction quality assurance (CQA) requirements for owners or operators of hazardous waste management facilities to help assure that land disposal units perform as designed.

### 1.1.2.2 Major Changes in Hazardous Waste Management

The HSWA amendments regarding hazardous waste are comprehensive and are requiring changes in the way that the United States manages its hazardous wastes. They are also significant because of the ambitious schedules Congress established to meet the requirements. For instance, minimum technology requirements relating to leak detection systems at landfills, surface impoundments, and waste piles, and land treatment units are required to be implemented within 30 months of the HSWA enactment (Nov. 8, 1984).

Some major changes required by the HSWA in hazardous waste management include:

- Minimum technology requirements for landfills, surface impoundments, and waste piles.

- Requirements for retrofitting certain existing surface impoundments with liners.

- Authority for EPA to expedite permits for new and innovative treatment technologies to foster research and development.

- A new program for identifying the health risks presented by surface impoundments and landfill facilities.

- Expanded requirements for ground-water monitoring and cleanup at permitted facilities, including land treatment units.

- A new program for banning wastes from land disposal.

## 1.2 PURPOSE AND SCOPE OF LINER/LEAK DETECTION RULE

The purpose of the set of regulations being proposed by EPA through its "liner/leak detection rule" is to continue implementation of the Congressional directives regarding hazardous waste management set out in the Resource Conservation and Recovery Act amendments of 1984 (RCRA 3004(o)). The objectives are being implemented through the liquids management strategies EPA has instituted for each type of land disposal unit under Code of Federal Regulations, Subparts K through N. In general, the liquids management program is a systematic effort to prevent to the greatest extent possible (using current technology) migration of hazardous constituents out of the unit.

### 1.2.1 Background

The new regulations proposed in the liner/leak detection rule should be considered as companions to the proposed changes in the minimum technology double liner system requirements presented in the Federal Register on March 28, 1986 (51 FR 10706). The Agency's proposed rule codification of March 28, 1986 requires the following double liner system standards at surface impoundments and landfills:

- A top liner designed, operated and constructed to prevent migration of liquids into it; and,

- One of two possible bottom liners;

  --A bottom liner designed, operated and constructed so that liquids do not migrate through it. The minimum standard is a 3-foot-thick layer of compacted soil with a maximum hydraulic conductivity of 1 x $10^{-7}$ centimeters per second (cm/s);

OR

  --A composite bottom liner made of two components. The upper component would be designed, operated and constructed to prevent migration of hazardous constituents into it and a lower component designed, operated and constructed to minimize migration of hazardous constituents through the upper component if the upper component were breached before the post-closure care period ends. This lower bottom liner component must be a compacted soil with a maximum hydraulic conductivity of 1 x $10^{-7}$ cm/s.

Subsequent to the proposed rule codification, EPA issued a Notice of Data Availability on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments" (April 17, 1987) which concluded that compacted low-permeability soil bottom liners can impede the proper functioning of a leak detection system. The Notice showed that in comparison to composite bottom liners, a compacted soil bottom liner:

- does not maximize leachate removal in the LCRS between the liners because the compacted soil will absorb some of the liquid from the LCRS;

- does not allow detection of leakage through the top liner at the earliest practicable time; and

- does not detect small amounts of leakage through the top liner.

For the above reasons, compacted soil bottom liners are believed to be inferior to composite bottom liners and do not represent best demonstrated available technology (BDAT). EPA is considering the possibility of eliminating the single compacted soil bottom liner as an option.

### 1.2.2 Leak Detection Systems

The Hazardous and Solid Waste Amendments require an "approved leak detection system" to be utilized at new landfills, surface impoundments, waste piles, and land treatment units. The leak detection system must be capable of detecting leakage at the "earliest practicable time over all areas likely to be exposed to waste and leachate during the active life and post-closure care period of the unit". The basis for the proposed leak detection system is the leachate collection and removal system (LCRS) between the top and bottom liners as required by the Proposed Codification Rule of March 28, 1986 (51 FR 10707-12). The Proposed Codification Rule requires a LCRS between the liners that is designed, constructed, maintained and operated to detect, collect, and remove liquids that leak through any area of the top liner during the active life and post-closure care period. The proposed rule further requires that the LCRS between the liners be constructed of materials that are chemically resistant to the waste or leachate in the unit and be designed and operated to function without clogging during the active life and post-closure care period of the unit.

### 1.2.3 Extension of Minimum Technology Standards

#### 1.2.3.1 Waste Piles

A double liner system meeting the minimum technology requirements is proposed for all newly-constructed waste piles. This is not specifically mandated by HSWA, but it is being proposed to satisfy the understood intent of Congress for equivalent levels of protection of human health and the environment for all units manifesting similar potentials for migration of hazardous constituents out of the unit.

#### 1.2.3.2 Significant Portions

A significant portion is any unlined area of a unit that has not received waste and, if double lined before receiving waste, would significantly reduce the potential for migration of hazardous constituents out of the unit, thereby reducing the potential for ground-water and surface-water contamination.

A double liner system that meets the minimum technology requirements of RCRA 3004(o) is proposed for all significant portions of existing landfills, surface impoundments, and waste piles. This specific provision is not mandated by HSWA, but is proposed for the purpose of equivalence.

### 1.2.4 Construction Quality Assurance Program

This program is proposed to be added to the hazardous waste management regulations to minimize leachate migration into the environment by first ensuring the effectiveness of the facility's design, construction and operating plan. A construction quality assurance program is proposed for all permitted and interim status landfills, surface impoundments, waste piles, and land treatment units. This would apply to all units where construction begins after promulgation of the liner/leak detection rule.

### 1.2.5 Applicable Units

These proposed regulations apply to landfills, surface impoundments, waste piles, and land treatment units. Underground tank leak detection rules were promulgated under RCRA 3004(o)(4) and 3004(w) on July 14, 1986.

## 1.3 PURPOSE AND SCOPE OF THE BACKGROUND DOCUMENT

### 1.3.1 Purpose of the Background Document

The purpose of this document is to provide the technical rationale and support for the three main portions of the proposed Liner/Leak

Detection Rule: (1) leak detection system requirements; (2) extension of the double-liner system requirements to waste piles, significant unused portions and certain other units; and (3) construction quality assurance program requirements.

The portion of this document devoted to leak detection (Chapter 2) will: (1) present the technologies and materials available to construct lining systems and leak detection systems to meet the proposed regulations; (2) recommend best demonstrated available technologies (BDAT) to meet the requirements stemming from these regulations; and (3) quantify leak detection system performance capabilities associated with BDAT.

This document, to the greatest extent possible, presents a state-of-the-art review of available technologies and achievable performance levels in leak detection systems at hazardous waste management units. This review is necessary to establish the best demonstrated available technology (BDAT) for leak detection systems. It provides the critical information considered in developing EPA's regulatory options and describes the criteria used to select among the options. The document also presents a rationale and technical data to support extension of the double-liner system requirements to waste piles, significant unused portions, and certain other units. Lastly, the document presents a comprehensive discussion of the issues, methodologies, and benefits associated with construction quality assurance programs at hazardous waste management facilities.

## 1.3.2 Scope of the Background Document

### 1.3.2.1 Technical Information Areas

#### 1.3.2.1.1 Leak Detection Systems

The proposed regulations will require leak detection systems for new units of landfills, surface impoundments, waste piles, and land treatment units. The technical areas to be discussed include: lining systems materials and performance; leak detection technologies; performance characteristics of leak detection systems between liners;

analyses of the functioning of leak detection systems; action leakage rate; and response action plans. This information is presented in Chapter 2.

#### 1.3.2.1.2 Double Liner System for New Waste Pile Units

The proposed regulations will require minimum technology double liner systems at all newly-constructed waste piles. The technical areas discussed include the description and number of waste piles; the rationale for the proposed changes; the comparable risks to landfills; and the issue of equivalent protection. This information is presented in Chapter 3.

#### 1.3.2.1.3 Significant Portions

The proposed regulations will require minimum technology double liner systems at significant unused portions of existing landfills, surface impoundments, and waste piles. The technical discussion of this proposed regulation includes a description of the number of units it will affect; the rationale for the proposed changes; and the issue of equivalent protection. This information is presented in Chapter 3.

#### 1.3.2.1.4 Double Liners for New Units, Replacements, and Lateral Expansions of Facilities Permitted Prior to November 8, 1984

The proposed regulations will require minimum technology double liner systems at new units, replacements, and lateral expansions of landfills, surface impoundments, and waste piles at facilities permitted prior to November 8, 1984 (except for certain replacement units permitted prior to November 8, 1984). The technical areas to be discussed are the comparable risks to landfills and surface impoundments that these units represent; and the equivalent protection a double liner system will provide. This information will be presented in Chapter 3.

#### 1.3.2.1.5 Construction Quality Assurance Program

The proposed regulations will require a construction quality assurance program (CQA) for all permitted and interim status landfills, surface impoundments, waste piles and land treatment units regulated under 40 CFR Parts 264 and 265 for which construction commences later than 12 months after promulgation of the liner/leak detection rule. Technical issues to be discussed in this section of the background document include: rationale for a CQA program; components of a CQA plan; sensitivity of lining system performance to construction procedures; benefits of a construction quality assurance program; and testing procedures to evaluate construction quality. This information will be presented in Chapter 4.

#### 1.3.2.1.6 Land Treatment Units

The proposed regulations will require leak detection for all land treatment units. These requirements include a 95 percent confidence level of detection of hazardous constituents below the treatment zone, detection of leakage into the area below the treatment zone at the earliest practicable time, monitoring above the seasonal high water table, and a response action plan for widespread leakage. This information will be presented in Chapter 5.

# Chapter 2

# LEAK DETECTION

## 2.1 INTRODUCTION

### 2.1.1 Scope of Chapter 2

The purpose of Chapter 2 "Leak Detection" is to provide in a single document a complete discussion of issues, technologies, design and construction, and achievable performance levels for leak detection systems at surface impoundments, landfills, and waste piles. This chapter is intended to provide EPA with comprehensive technical background information on leak detection systems to support its rulemaking activities related to the proposed liner/leak detection rule.

In the Introduction (Section 2.1), the following general introductory material is provided: the definition of hazardous waste management units and lining systems used in these units; definition of leakage (what is leakage? what is a leak?); purpose of leak detection systems (why is it important to detect leakage? complementarity of leak detection and leachate collection); and performance characteristics of leak detection systems (such as ability to evaluate leakage rate, leak detection time, and determination of leak location). This section is included as background to ensure that all readers are familiar with the important basic concepts of hazardous waste management units, waste containment, lining systems, leachate collection and removal systems, and leak detection systems.

Section 2.2 is concerned with lining systems materials and performance. This section first summarizes information regarding materials used in lining systems. A discussion of the performance of lining systems is then presented. Failure mechanisms for geomembrane and soil liners are discussed. Case study information is presented on the frequency of seam defects observed in actual geomembrane installations. Calculations are presented to estimate leakage through holes in geomembranes and composite liners. The information presented in this section will be useful in establishing the best demonstrated available technology (BDAT) for top liner systems, from which action leak rate recommendations will subsequently be derived (Section 2.9).

Section 2.3 is devoted to leak detection technologies and includes two parts. The first part is a review of available technologies and includes: leachate collection and removal systems used as leak detection systems, systems based on acoustic emissions, electric resistivity, and time domain reflectometry, and other technologies. The second part of Section 2.3 provides a rationale for the selection of leachate collection and removal systems (LCRS) as leak detection systems. Thus, the remaining eight sections of Chapter 2 deal exclusively with leachate collection and removal systems which are used to satisfy the statutory requirements of RCRA for leak detection systems capable of detecting leakage in the shortest practicable time.

Section 2.4 provides a detailed description of LCRS type leak detection systems. Performance requirements, materials used to construct the LCRS, and the properties of these materials are covered. In particular, this section compares the properties of natural and synthetic drainage layer materials. This section is included as background to ensure that all readers have a source of basic information on drainage layer materials and properties to draw upon in the latter sections of Chapter 2 (when leak detection system performance standards and minimum drainage layer component specifications are discussed).

Sections 2.5 and 2.6 are concerned with the performance of lining systems and leak detection systems. This information is important to establish BDAT standards for leak detection systems. The information will be useful in developing recommendations for leak detection system performance criteria (Section 2.7) and minimum component specifications (Section 2.8), and for establishing the action leakage rate (Section 2.9). The information presented in Section 2.5 was obtained from field performance data (leakage collected in sumps). Section 2.6 presents the results of numerical simulations of lining system performance carried out by Radian Corporation and GeoServices Inc.

Section 2.7, entitled "Performance Criteria for Leak Detection Systems", provides recommendations on BDAT standards for leak detection systems. Recommendations are provided for minimum criteria for leak detection sensitivity, leak detection time, leachate collection efficiency, and design life of leak detection systems. These recommendations are based on the review of performance capabilities presented in Sections 2.5 and 2.6.

Section 2.8 is devoted to design specifications for the components of a leak detection system. Minimum specifications for the hydraulic conductivity and the hydraulic transmissivity of the materials used to construct the leak detection system are discussed, as is the minimum bottom slope requirement for the waste management unit and minimum properties for the leak detection system sump. The rationale for recommending certain minimum values is also discussed in this section.

Section 2.9 discusses the concept of an "action leakage rate (ALR)" which is the leakage rate through the top liner beyond which EPA intends to require an owner or operator of a unit to initiate response actions at the unit. ALR recommendations are suggested based on the BDAT top liner capabilities discussed in Sections 2.2 and 2.6.

Section 2.10 discusses the concept of a "response action plan", i.e., the actions that should be undertaken when the measured leakage rate exceeds the "action leakage rate" defined in Section 2.9. In Section 2.10 a discussion is provided on the important variables to consider in establishing appropriate response actions.

## 2.1.2 Waste Management and Land Disposal Units

### 2.1.2.1 Introduction

#### 2.1.2.1.1 Definition

"Waste management unit" is a generic term which encompasses all containment facilities used to treat, store, or dispose of hazardous waste. A type of waste management unit is a land disposal unit used for the temporary storage or permanent burial of hazardous waste.

Land disposal units include surface impoundments, waste piles, and landfills. In this document the term "land disposal unit" will be used as a term to identify surface impoundments, waste piles and landfills. To a lesser extent, the term "waste management unit" will be used to identify these units (although in its general usage, waste management unit includes other types of facilities as well).

### 2.1.2.1.2 Purpose of this Section

It is not possible to discuss leakage without a knowledge of:

- the containment facilities from which leakage is taking place; and
- the lining systems through which leakage is taking place.

The purpose of Section 2.1.2 is to briefly describe hazardous land disposal units, and to discuss pollution mechanisms that may be associated with leakage from these units. The next section (2.1.3) will be devoted to lining systems used in those units.

## 2.1.2.2 Description

### 2.1.2.2.1 Types of Land Disposal Units

Three types of land disposal units are considered in this document: landfills, surface impoundments, and waste piles. These three types of units are illustrated in Figure 2.1-1 and their usage is as follows:

- landfills are used for permanent disposal of solid waste (hazardous waste in "hazardous waste landfills" or municipal waste in "sanitary landfills");
- surface impoundments are used to store liquids (with, possibly, particles in suspension, which settle progressively) or sludges (which consolidate progressively); and

- waste piles are used for temporary storage of solid waste.

### 2.1.2.2.2 Geometry of Land Disposal Units

Surface Impoundments - The overall shape of surface impoundments is roughly that of an inverted truncated pyramid with "side slopes" and a "bottom". The side slopes can be as steep as permitted by geotechnical considerations and they typically range between 2H/1V and 4H/1V, while the bottom is nearly horizontal with just the slope (e.g., 2%) required for the drainage layer if there is a double liner.

Landfills - The lower part of a landfill has roughly the shape of an inverted truncated pyramid, like a surface impoundment. This is the part of a landfill that is lined prior to waste placement. The side slopes of the bottom part of a landfill can be as steep as permitted by geotechnical considerations and they typically range between 2H/1V and 4H/1V, while the bottom is nearly horizontal with just the slope (e.g., 2%) required for the drainage layer(s) that is (are) incorporated into the lining system.

The upper part of a landfill includes a cap that is placed on top of the waste to close the landfill after completion of waste placement operations. The cap is a lining system used to prevent (or, at least, minimize) penetration of rain water into the landfill.

Large landfills may be divided into cells which are operated sequentially.

Waste Piles - A waste pile can have any shape compatible with waste stability. The lining system placed under the waste pile is nearly horizontal, with just the slope (e.g., 2%) required for the drainage layer(s) that is (are) incorporated into the lining system.

### 2.1.2.3 Ground Pollution Mechanism

#### 2.1.2.3.1 Surface Impoundments

A surface impoundment can cause pollution of soil and ground water if the hazardous liquid contained in the impoundment leaks through the lining system and into the ground.

In rare occasions, waves of liquids stored in surface impoundments have overtopped the crests of the impoundments thereby causing groundwater pollution.

#### 2.1.2.3.2 Landfills

The mechanism by which a landfill can cause soil and ground-water pollution includes two steps:

- first, leachate is generated in the landfill; and
- then, pollution occurs if some leachate migrates through the lining system into the ground.

Leachate can be produced by two mechanisms, intrusion of water into the waste and generation of leachate within the waste:

- Intrusion of Water in the Waste. The main cause of leachate production is infiltration of rain water into the waste. The rain water seeping through the waste becomes progressively polluted and the resulting polluted liquid is called "leachate". In exceptional cases, leachate can be produced by intrusion of ground water into the waste (if the ground water table rises), or, even more exceptionally, by intrusion of flood water into the waste.
- Generation of Leachate within the Waste. Leachate can originate in the waste if liquid is entrapped in the waste during waste placement. Drums containing liquids are not allowed in hazardous waste landfills, and the only possibility

for entrapping liquids is through moisture in the waste or in the earth used for the daily covers (i.e., the layers of compacted earth, placed every day on the waste). Part of the moisture included in the waste or the daily covers can be expelled by consolidation (i.e., decrease in volume of the waste and the daily covers due to compression caused by the own weight of the waste and the daily covers themselves).

To prevent pollution of soil and ground water by landfills, all efforts should be made to prevent production of leachate:

- A low-permeability cap must be placed on the landfill immediately after completion of waste placement operations to prevent intrusion of rain water.
- Selection of landfill location and appropriate design should minimize intrusion of ground water and flood water.
- Waste and daily cover material should not contain excess liquids.

Since leachate production cannot be totally prevented, especially during landfill operation (i.e., during waste placement) when rain can fall freely on the landfill, a lining system is necessary at the bottom and on the side slopes of the landfill.

### 2.1.2.3.3 Waste Piles

The two-step mechanism by which waste piles can cause soil and ground-water pollution is similar to the mechanisms related to landfills which were described in Section 2.1.2.3.2. Waste piles are temporary storage units and the waste is normally removed after some time.

## 2.1.3 Lining Systems Used in Land Disposal Units

### 2.1.3.1 Introduction

#### 2.1.3.1.1 Importance of Lining Systems

From the above discussion, it is clear that the lining system placed on the bottom and the side slopes of a land disposal unit has a critical role: the ground is polluted as soon as liquid leaks through the lining system. Therefore it is essential to have a good knowledge of lining systems prior to discussing leakage.

#### 2.1.3.1.2 Scope of this Section

The purpose of this section is to provide basic information on the types of lining systems used in hazardous waste land disposal units, and on the materials used to construct these lining systems. This section should familiarize the reader with the vocabulary used to describe lining systems.

This section will address the following: definition of lining systems, materials used in lining systems, double liners, and composite liners. (Experience shows that it is not practical to discuss double liners and composite liners without a knowledge of materials used to construct lining systems.)

#### 2.1.3.1.3 Definition of Lining Systems

The terms "liner" and "lining system" are not synonymous.

A liner is a low-permeability barrier used to impede liquid or gas flow. Note that "low permeability" is used, and not "impermeable". If there was such a thing as an impermeable barrier, it would be possible to prevent leakage, and many of the discussions and considerations presented in this background document would be pointless. Although it may be possible that a glass is impermeable to water, in modern technology there is no material that is impermeable at the scale of a land disposal unit where the area to be lined can be as large as tens of hectares (dozens of acres).

Since no liner is impermeable, pollution control can only result from a combination of liners and drainage layers, performing complementary functions:

- Liners (which are low-permeability barriers) impede the flow of undesirable (polluted) liquids toward the ground.
- Drainage layers (which have a high permeability) convey the undesirable flow away from the ground.

Such combination of liners and drainage layers is called a "lining system".

### 2.1.3.2 Materials Used in Lining Systems

#### 2.1.3.2.1 Introduction

- low-permeability materials to construct the liners;
- high-permeability materials to construct the drainage layers;
- transition materials (or interface materials) acting as filters or protective layers (i.e., providing filtration or protection) between various layers of a lining system; and
- reinforcement materials which increase the strength of a lining system (if required).

These materials are briefly discussed below.

### 2.1.1.2.2 Liner Materials

Low-permeability materials used in civil engineering to construct liners include: compacted low-permeability soils, geomembranes, concrete, and asphaltic concrete. Concrete and asphaltic concrete are not used in hazardous waste units for the following reasons:

- Concrete liners tend to undergo cracking and therefore tend to leak significantly.

- Asphaltic concrete cannot be used because asphalt has a poor

Therefore, only low-permeability soils and geomembranes are discussed in this document.

Compacted Soils - Compacted low-permeability soils used to construct liners include: clay, silty clay, clayey sands, and silty sands. If such soils are not available at the site, it is possible to make a compacted low-permeability soil by mixing bentonite with sand. Bentonite is composed of extremely small particles of sodium montmorillonite. When it is dry, it becomes a powder which can be put in bags, and is purchased and transported like cement.

In the remainder of this document compacted low-permeability soil liners will be referred to simply as compacted soil liners.

Geomembranes - Geomembranes are low-permeability membranes used in civil engineering as fluid barriers. By definition, a membrane is a material that is thin and flexible.

All geomembranes presently used in hazardous waste management units are synthetic geomembranes. (Asphaltic geomembranes, which are used for lining water storage facilities, are not used in hazardous waste containment units because they do not have adequate resistance to chemical attack.) Typical examples of geomembranes used in hazardous waste containment units include: high density polyethylene (HDPE) geomembranes; linear low density polyethylene (LLDPE) geomembranes; polyvinyl chloride (PVC) geomembranes; and chlorosulfonated polyethylene (CSPE) geomembranes.

The term geomembrane is often used by the engineering community in place of the term "flexible membrane liner" (FML). EPA is using the term "flexible membrane liner" or "FML" to be consistent with the terminology used in the past in documents discussing waste management units. Therefore, for consistency with previous EPA documentation, "flexible membrane liner" or "FML" will be used in the remainder of this document to describe synthetic membranes used as low-permeability liners.

### 2.1.3.2.3 Drainage Materials

High-permeability materials used to construct drainage layers include: high-permeability soils, synthetic drainage materials, and pipes. High-permeability soils and synthetic drainage materials are discussed below.

High-Permeability Soils - High-permeability soils include a wide variety of sands and gravels ranging from fine to coarse in size and well-graded to uniform in gradation. Selection of a high-permeability soil for specific conditions must consider the following:

- the drainage layer should be able to collect and rapidly remove liquids entering the leak detection, collection and removal system as a result of leakage through the top liner;

- the high-permeability soils should not damage FMLs when the FMLs are directly in contact with the soils; and

- the drainage layer should be physically compatible with transition materials to prevent any potential migration of the transition materials into the drainage layer which could lead to clogging.

Synthetic Drainage Materials - Synthetic drainage materials are made of planar structures which are thick enough to convey fluids in their plane. Synthetic drainage materials are usually made from polymers. Typical polymers include polypropylene, polyester, and polyethylene. These polymers are highly inert to biological and chemical degradation.

Four types of synthetic drainage materials are currently available. These are thick needlepunched nonwoven geotextiles, geonets, geomats and corrugated or waffled plates. With the exception of needlepunched nonwovens, these materials can be combined with geotextile filters to form drainage geocomposites.

### 2.1.3.2.4 Transition Materials

Transition materials include filters or protective layers.

Filters - Filters are located between the drainage layer and the soil to be protected. They usually consist of a granular layer or a combination of granular layers, or a geotextile. Their function is to allow free flow into the drainage layer and at the same time prevent the migration of particles of the protected soil into the drainage layer.

Protective Layers - Protective (cushion) layers are located between the drainage layer and the FML. Their function is to protect the FML from damage by the drainage material. Cushion layers usually consist of a sand layer or a thick needlepunched nonwoven geotextile.

### 2.1.3.2.5 Reinforcement Materials

Reinforcement materials are typically placed in a soil layer. Typical functions include reinforcing the lining system on steep slopes to prevent sliding along the slope, reinforcing slopes to prevent slope failure, or bridging over cavities, depressions or soft spots. The materials most frequently used in reinforcement applications at waste management units are geogrids and geotextiles.

## 2.1.3.3 Double Liners

### 2.1.3.3.1 Introduction

- Double Liner

A "double liner lining system" simply called a "double liner system" or a "double liner" is a lining system which includes two liners with a leachate collection and removal system between the two liners.

Clearly, two liners in contact (i.e., without a leachate collection and removal system between the two liners) do not constitute a double liner (they constitute a single liner, as discussed below).

- Single Liner

A lining system which includes only one liner is called a "single liner".

- Composite Liner

A composite liner is a liner comprised of two or more low-permeability components made of different materials in contact with each other. For example, a FML and a compacted soil layer placed in contact with each other constitute a composite liner. Composite liners do not constitute a double liner because there is no leachate collection and removal system between the two low-permeability components.

The purpose of a FML-compacted soil composite liner is to combine advantages of FMLs and soils. FMLs have a much lower permeability than compacted soils, but they may have holes through which leakage can occur if the FML is placed on a pervious medium and then subjected to a hydraulic head on its top surface. The leakage rate through a FML hole is reduced if there is compacted low-permeability soil under the FML.

- Terminology Related to the Liners

In this document, the upper liner of a double liner is called "top liner" and the lower liner is called "bottom liner". We recognize that this terminology may be confusing since the term "bottom liner" may be mistaken for "bottom lining system", i.e., the lining system located at the bottom of a waste management unit.

"Top liner" is synonymous with "upper liner" or "primary liner".

"Bottom liner" is synonymous with "lower liner" or "secondary liner".

- Terminology Related to the Leachate Collection and Removal Systems

In all land disposal units lined with a double liner there is a pervious layer between the two liners. This layer is called the "leachate collection and removal system (LCRS) between the liners". If this system is also used as a leak detection system (LDS), its name becomes "leak detection, collection, and removal system" (LDCRS).

While in surface impoundments there is only one pervious layer (i.e., the LDCRS mentioned above), there are two pervious layers in landfills: the LDCRS and the layer located above the top liner and called the "leachate collection and removal system (LCRS) above the top liner".

### 2.1.3.4 Use of Double Liners in Land Disposal Units

#### 2.1.3.4.1 Current Regulations

Current EPA regulations (40 CFR Parts 264 and 265) require a double liner system in all new hazardous waste landfill and surface impoundment units except units permitted prior to November 8, 1984 or units where variances are allowed. Furthermore, as discussed in Chapter 1, the two liners comprising the double liner system should meet the following requirements:

- "A top liner designed, operated, and constructed of materials to prevent the migration of any constituent into such liner during the period such facility remains in operation (including any post-closure monitoring period)".

- "A bottom liner designed, operated and constructed to prevent the migration of any constituent through such liner during such period. For the purpose of the preceding sentence, a lower liner shall be deemed to satisfy such requirement if it is constructed of at least a 3-foot thick layer of recompacted clay or other natural material with a permeability of no more than 1 x $10^{-7}$ centimeter per second."

According to the Draft Minimum Technology Guidance on Double Liner Systems of May 24, 1985 (see EPA/530-SW-85-012):

- The top liner FML should be at least 0.75 mm (30 mil) thick, if it is protected in a timely manner after placement; if it is not protected in a timely manner the top liner FML should be at least 1.15 mm (45 mil) thick.

- The upper FML component of a bottom composite liner should be at least 0.75 mm (30 mil) thick.

### 2.1.3.4.2 Examples of Uses of Double Liners in Land Disposal Units

- Types of Double Liners Used in Land Disposal Units

From the above discussion, it appears that four types of double liners are currently allowed by existing EPA regulations (Figure 2.1-2). Such double liners can be used for landfills, surface impoundments, and waste piles. The double liner using two composite liners (Figure 2.1-2(b)), called "double composite liner", is increasingly used in order to minimize the amount of leakage through the top liner while maximizing the collection efficiency of the LDCRS.

- Caution on the Use of Top Composite Liners in Surface Impoundments

The use of a top composite liner in a surface impoundment requires special caution. If the FML (which is the upper component of the top composite liner) is not covered with a heavy material (such as a layer of earth, or concrete slabs), and if there is leakage through the FML, liquids tend to accumulate between the low permeability soil (which is the lower component of the top composite liner) and the FML since the submerged portion of the FML (whose specific gravity is close to 1) is easily uplifted. Then, if the impoundment is rapidly emptied, the FML is subjected to severe tensile stresses because the pressure of the entrapped liquids is no longer balanced by the pressure of the impounded liquid. Therefore, a top composite liner should always be loaded, which is automatically the case in a landfill or in a waste pile, and which must be taken into account in the design of a liquid impoundment.

### 2.1.3.4.3 Influence of Top and Bottom Liners on Leak Detection

The LCRS between the top and bottom liner is also used as a leak detection system to form a leak detection, collection, and removal system (LDCRS). The leakage through the top liner flows in the LDCRS over the top surface of the bottom liner. The two liners have the following influence:

- The top liner governs the amount of leakage entering the LDCRS. Many hazardous waste management units include a top composite liner in order to minimize leakage through the top liner.

- The bottom liner has a major influence on the performance of the LDCRS. As was shown in EPA's recent Notice of Availability of Data on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments" (US EPA, 1987), a compacted soil liner allows greater leakage into and through the bottom liner than does a composite. For this reason, a composite bottom liner (Figure 2.1-2 (a and b)) is preferable to a compacted soil bottom liner (Figure 2.1-2 (c and d)).

As was shown in the Notice of Availability of Data, owners and operators of hazardous waste management units rarely use compacted soil bottom liners (Figure 2.1-2 (c and d)) because of the performance deficiencies associated with them in comparison to composite bottom liners (Figure 2.1-2 (a and b)).

## 2.1.4 Leakage Definition and Detection

### 2.1.4.1 Definitions

#### 2.1.4.1.1 Leak and Leakage

According to Webster:

- A leak is "a crack or opening that permits something to escape from or enter a container or conduit".

- Leakage is "something that escapes by leaking" or "an amount lost as the result of leaking".

From these definitions, it clearly appears that what is monitored between the top and the bottom liners is the leakage, not the leaks. Therefore, the monitoring system should be called "leakage detection system". While "leakage detection system" is grammatically correct, the phrase "leak detection system" has been codified by RCRA. For the sake of consistency with the law, the phrase "leak detection system" will be used in this document.

On the other hand, systems used in quality assurance of geomembrane installation, such as the vacuum box, are clearly intended to find leaks.

### 2.1.4.1.2 Leak Size and Leakage Rate

According to the above definitions, the term "leak size" designates the size of a hole, expressed as a surface area or dimensions such as a diameter (e.g., a 1 $cm^2$ leak, a 1 $in.^2$ leak, a 2 mm diameter leak, a 1/4-in. diameter leak). The term "leak size" is sometimes mistakenly used for "leakage rate" which is the flow rate through a leak or a group of leaks, which is expressed as a volume per unit of time ($m^3/s$, liters/day, gallons/day). The term "leakage rate" will often be used in this document as an abbreviation for "leakage rate per unit area", which is expressed as a volume per unit of time per unit of area ($m^3/s/m^2$ (which is equivalent to m/s), liters/hectare/day, liters/1000$m^2$/day (Ltd), gallons/acre/day (gpad)). (Note: 1 hectare = 100 m x 100 m = 10 000 $m^2$.)

The following conversions apply:

1 gallon/acre/day = $1.08 \times 10^{-11}$ m/s
= 9.35 liters/hectare/day
= 0.935 liters/1000 $m^2$/day

1 liter/hectare/day = $1.16 \times 10^{-12}$ m/s
= 0.11 gallons/acre/day
= 0.1 liters/1000 $m^2$/day

1 liter/1000$m^2$/day = $1.16 \times 10^{-11}$ m/s
= 1.1 gallon/acre/day
= 10 liters/hectare/day

1 m/s = $8.64 \times 10^{10}$ liters/1000$m^2$/day
= $8.64 \times 10^{11}$ liters/hectare/day
= $9.24 \times 10^{10}$ gallons/acre/day

From a practical standpoint, the approximate conversion can be used:

1 liter/1000$m^2$/day = 1 gallon/acre/day
1 Ltd = 1 gpad

### 2.1.4.1.3 Leakage Collected and Leakage Out of the Unit

The possible fates of liquids entering a double liner system are shown in Figure 2.1-3.

The leakage discussed in the previous sections is the leakage that the LDCRS system is intended to collect and detect. This is the leakage through the top liner (C in Figure 2.1-3).

The leakage out of the unit, which is the leakage through the bottom liner (J in Figure 2.1-3), is only a fraction of the leakage through the top liner. Other fractions include:

- leakage entrapped in the LDCRS by absorption, capillarity, ponding, etc. (F in Figure 2.1-3);
- leakage collected at the LDCRS sump (G in Figure 2.1-3); and
- leakage absorbed in the bottom liner (I in Figure 2.1-3).

If the LDCRS is properly designed, the liquid head on the bottom liner is very small, and leakage through the bottom liner (which is governed by head on the bottom liner) is very small. This is consistent with EPA's goal of protecting human health and environment through system impermeability and not liner impermeability. No liner is perfectly impermeable but proper design can almost achieve system impermeability.

### 2.1.4.2 Leak Detection System

#### 2.1.4.2.1 Definition

In the context of this background document, leak detection refers to leakage through the top liner and, therefore, a leak detection system is a system which is placed between the two liners of a double liner system to monitor the leakage through the top liner.

#### 2.1.4.2.2 Purpose of Leak Detection

As indicated in the above definition, the purpose of a leak detection system is to monitor leakage through the top liner. Monitoring leakage through the top liner is an important component to EPA's systems approach to the containment of hazardous constituents using double liner systems.

#### 2.1.4.2.3 Performance Requirements of Leak Detection Systems

- Review of Potential Performance Requirements

What performance requirements should be considered when designing or evaluating a leak detection system? In other words, what do we expect from a leak detection system.

Based on the discussion presented previously it can be deduced that the leak detection system should essentially:

- evaluate the leakage rate; and
- provide this information rapidly so action can be taken without delay, if necessary.

Leak detection systems may also perform other functions such as:

- leachate collection; and
- leak location identification.

If the drainage layer placed between the top and bottom liners is used for leak detection, it collects leakage as well as detects it. On the other hand, electric resistivity or acoustic emission leak detection systems detect leakage without collecting it. It thus appears that leakage detection and collection are two separate functions (although, in some cases, they can be performed simultaneously). Leachate collection is already provided in all hazardous waste management units because there is a statutory requirement for a leachate collection and removal system between the two liners. Therefore, the ability to collect leakage cannot be considered as a performance characteristic for a leakage detection system. However, it can be considered as an additional benefit.

The need to locate a leak in a land disposal unit will vary based on factors such as the type of unit, stage of active life, rate of leakage and available response actions. In many instances, the need to locate a leak will be limited because:

- In landfills, no significant leakage is likely to occur after closure of the unit. If the rate of leakage through the top liner is in the range of a typical remedial action consists of capping a certain area of the unit (i.e., covering the waste with a liner) to prevent infiltration of rain water, thereby substantially reducing leachate production. It is not necessary to know exactly where leakage occurs to design the capping of an area of the unit.
- In surface impoundments, the strategy in the case of a large leak is different. The impoundment can be emptied and the liner either repaired or replaced (retrofitted). It is therefore useful to know the location of the leaks. However, leaks can be located by means that do not need to be built into

the lining system. These means include portable electric resistivity equipment, vacuum testing, or visual inspection of the lining system after emptying of the impoundment.

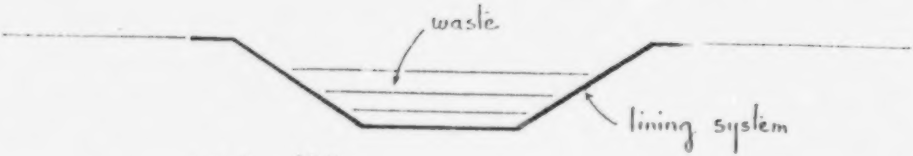
- Selection of Performance Requirements for Leak Detection Systems

From the above discussion, it appears that the two key performance characteristics that should be considered when designing or evaluating a leak detection system are:

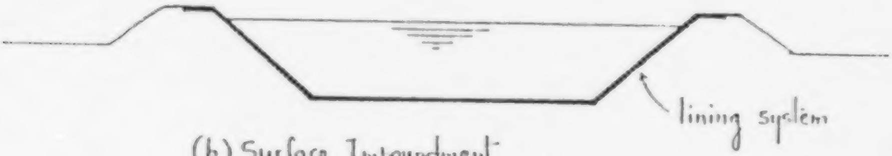
- the ability of the system to correctly evaluate leakage rate; and

- the ability of the system to provide rapid information on leakage rate.

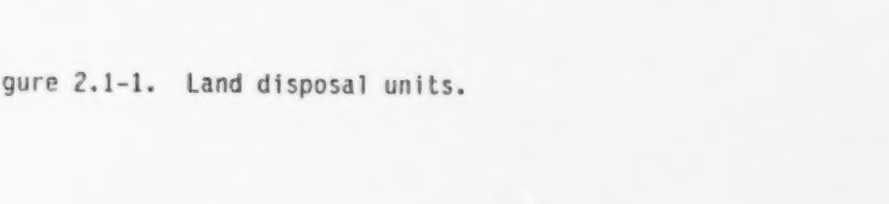
It also appears from the discussion that: (i) the ability of the system to collect leachate is not a relevant performance characteristic for leak detection systems (however it is the key performance characteristic for leachate collection and removal systems, and since the LDCRS serves as both a leak detection system and a leachate collection and removal system the LDCRS must have leachate collection and removal capability); and (ii) the ability of the system to locate leaks is not a primary performance characteristic.



(a) Landfill



(b) Surface Impoundment



(c) Waste Pile

Figure 2.1-1. Land disposal units.

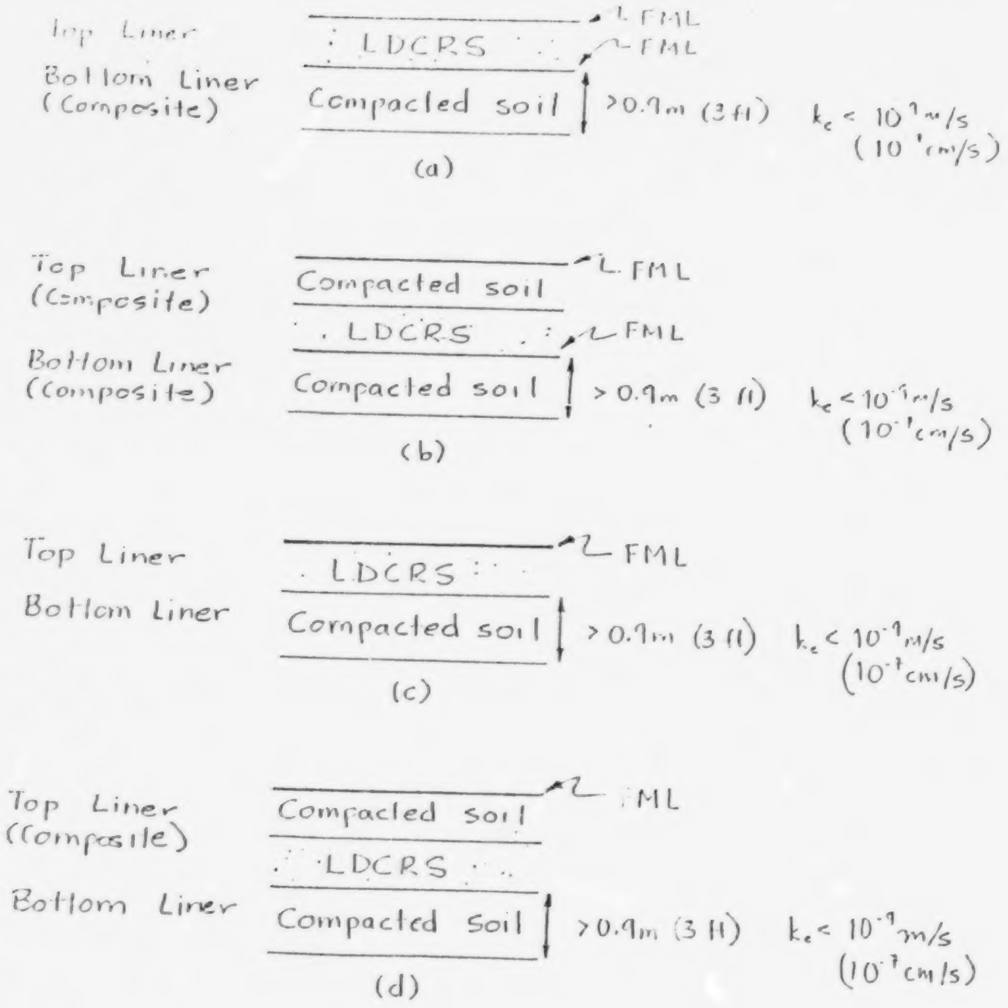
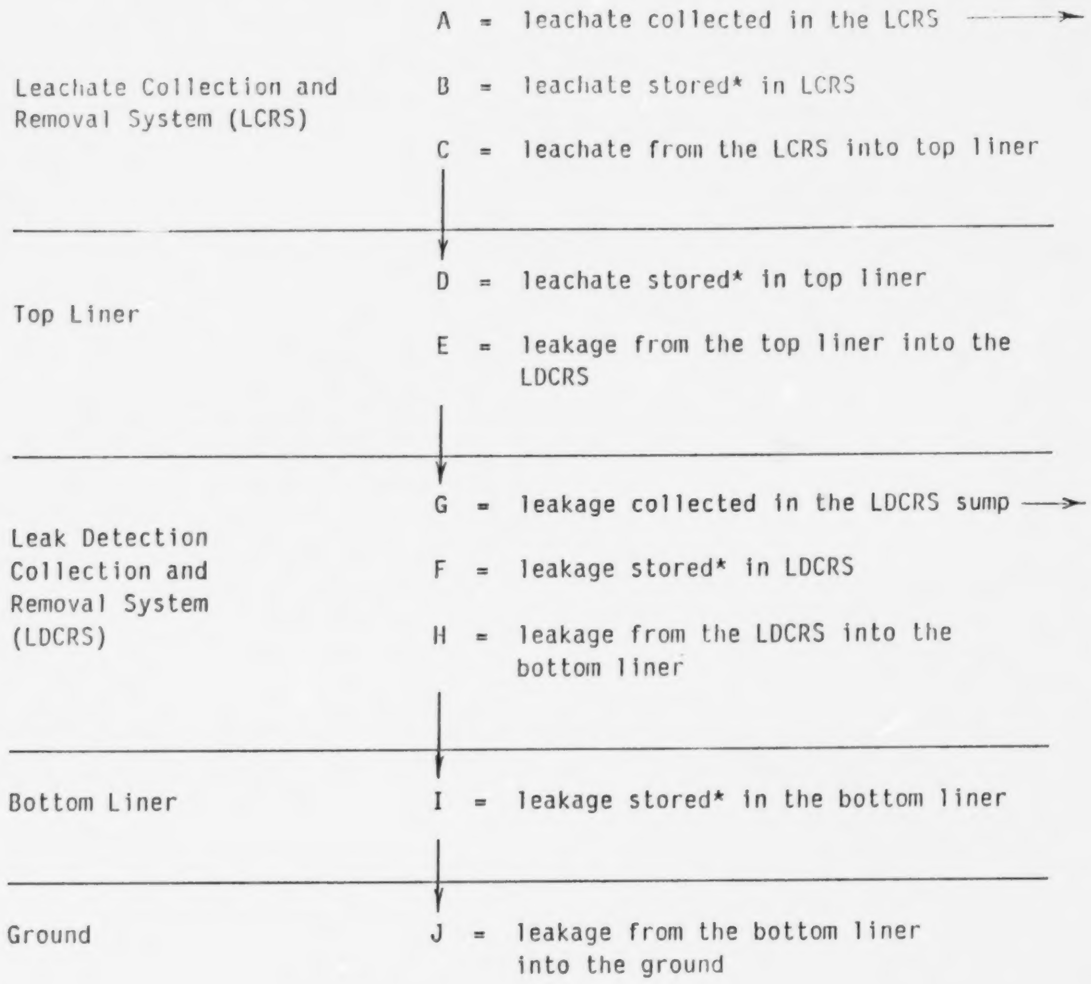


Figure 2.1-2. Four types of double liner systems used in hazardous waste land disposal units.


A = leachate collected in the LCRS
Leachate Collection and Removal System (LCRS)
B = leachate stored* in LCRS
C = leachate from the LCRS into top liner
D = leachate stored* in top liner
Top Liner
E = leakage from the top liner into the LDCRS
G = leakage collected in the LDCRS sump
Leak Detection Collection and Removal System (LDCRS)
F = leakage stored* in LDCRS
H = leakage from the LDCRS into the bottom liner
Bottom Liner
I = leakage stored* in the bottom liner
Ground
J = leakage from the bottom liner into the ground

* Stored liquids due to capillarity, absorption, etc.

Figure 2.1-3. Fate of liquids entering a double liner system at a landfill unit.

## 2.2 TOP LINER PERFORMANCE

### 2.2.1 Introduction

#### 2.2.1.1 Scope

It is not possible to discuss leakage detection without first discussing leakage through top liners since the leakage detected by the leak detection system is the leakage that has migrated through the top liner, as indicated in Sections 2.1.4.1.3 and 2.1.4.2.1. Also, it is necessary to understand the information presented in this section concerning how well top liners perform in order to establish recommendations for the action leakage rate (ALR) discussed in Section 2.9.

#### 2.2.1.2 Organization

Section 2.2 is organized as follows:

- Section 2.2.2 first presents the two types of top liners, i.e., the FML top liners and the composite top liners. Then, Section 2.2.2 discusses top liner materials, and presents an overview of typical defects likely to occur in top liners.

- Sections 2.2.3 reviews and analyzes data pertinent to leakage through FML top liners, while Section 2.2.4 does the same thing for composite top liners.

- Section 2.2.5 presents conclusions drawn from data presented in Sections 2.2.3 and 2.2.4.

## 2.2.2 Top Liners

### 2.2.2.1 Types of Top Liners

#### 2.2.2.1.1 FML

EPA's Proposed Double Liner Rule of March 28, 1986 (FR 10706-10723) requires a top liner designed, constructed, and operated to prevent migration of liquids into it. This requirement has been interpreted to mean that the top liner, at a minimum, must consist of a FML. According to the Draft Minimum Technology Guidance on Double Liner Systems of May 24, 1985 (EPA 530-SW-85-012), the FML top liner should be at least 0.75 mm (30 mil) thick, if it is protected in a timely manner after placement; if it is not protected in a timely manner, the top FML should be at least 1.12 mm (45 mil) thick.

#### 2.2.2.1.2 Low-Permeability Compacted Soil

Based on the above interpretation of EPA's Proposed Double Liner Rule, a layer of low-permeability soil alone (i.e., without an FML) is not accepted as a top liner for hazardous waste land disposal units, However, a layer of low-permeability compacted soil can be used in association with an FML to form a composite liner, as discussed below.

#### 2.2.2.1.3 Composite Liner

Composite liners were defined in Section 2.1.3.3.1 as liners comprised of two or more low-permeability components made of different materials in contact with each other. The composite liner used as a bottom liner for hazardous waste land disposal units in the Proposed Double Liner Rule must have an upper component designed, operated, and constructed to prevent migration of hazardous constituents into it and a lower component designed, operated, and constructed to minimize migration of hazardous constituents through it if the upper component were breached before the post closure care period ends. At a minimum, this lower bottom liner component must be a compacted soil with a maximum hydraulic conductivity of $10^{-9}$ m/s ($10^{-7}$ cm/s).

The EPA does not require that the top liner be a composite liner. However, composite top liners are accepted by the EPA and they are increasingly used by owners and operators in order to minimize leakage through the top liner. In addition, since composite top liners are not required, the low-permeability soil layer used in a composite top liner does not need to meet the hydraulic conductivity and thickness requirement indicated above.

### 2.2.2.2 Types of Materials Used for Top Liners

#### 2.2.2.2.1 FMLs

FMLs have been described in [USEPA, 1983] and [Giroud and Frobel, 1983] and the remainder of this Section is reproduced from the latter.

Polymers are chemical compounds of high molecular weight. Only synthetic polymers are used to make FMLs. The most common types of polymers presently used as base products in the manufacture of FMLs can be classified as indicated in Table 2.2-1. FMLs most often used in hazardous waste management units include HDPE, LLDPE, CSPE, and PVC (the latter mostly for caps on top of landfills).

FMLs may be non-reinforced or reinforced with a fabric. According to Giroud and Frobel [1983], fabric reinforcement is used for one or several of the following reasons: (1) to impart stability to the compound (e.g., asphalt, CSPE) during the manufacturing process; (2) to provide dimensional stability to compounds that would shrink or expand excessively as a result of change in environmental conditions such as temperature; (3) to increase the mechanical strength (tensile, tear, burst, puncture) of the FML to prevent it from being damaged during handling and installation, and to allow it to withstand design stresses; and (4) to increase the deformation modulus of the FML in order to decrease its elongation when subjected to tensile stresses.

Fabric reinforcement can be of various types depending on the manufacturing process of the FML. Fabric reinforcement used with FMLs are woven or nonwoven (typically needlepunched) fabrics. An increasing quantity of FMLs are reinforced with a nonwoven fabric placed on one side of the FML. However, most reinforced FMLs presently available are reinforced with a woven fabric placed inside the FML. The most typical case is a "scrim" (light-weight open weave fabric) placed between two plies of polymeric compound. Stability of the three-ply FML is ensured by adhesion of the two polymeric plies through openings of the scrim. This adhesion mechanism is called "strike-through".

Some FMLs have a roughened or embossed surface which increases their friction coefficient in order to prevent the development of slip surfaces on FMLs installed on slopes.

### 2.2.2.2.2 Low-Permeability Compacted Soils

While the types of low-permeability compacted soils that can be used in bottom composite liners are limited by the EPA requirement for a maximum hydraulic conductivity of 1 x $10^{-9}$ m/s (1 x $10^{-7}$ cm/s), a variety of low-permeability soils can be used to construct the low-permeability compacted soil layer of composite top liners. These low-permeability compacted soils include clays, silty clays, clayey sands, and silty sands.

## 2.2.2.3 Permeability of Liner Materials

### 2.2.2.3.1 Introduction

FMLs and low-permeability compacted soils are used to construct lining systems essentially because of their low permeability to liquids. However, these liner materials are not totally impermeable and, to discuss leakage, it is essential to have a good understanding of the permeability of liner materials.

### 2.2.2.3.2 Permeability of Compacted Soils

Flow rate of liquids through porous media such as soils can be expressed by Darcy's equation:

$$Q = k\ i\ A \qquad \text{(Equation 2.2-1)}$$

where: $Q$ = flow rate; $k$ = soil hydraulic conductivity; $i$ = hydraulic gradient; and $A$ = area perpendicular to the flow. Recommended SI units are: $Q$ ($m^3/s$), $k$ ($m/s$), $i$ (dimensionless), and $A$ ($m^2$).

The apparent flow velocity can be defined as the velocity the liquid would have if the liquid were flowing over the entire area. The apparent flow velocity is therefore obtained by dividing the flow rate by the area perpendicular to the flow:

$$v = Q/A = k\ i \qquad \text{(Equation 2.2-2)}$$

The liquid flows only in a fraction of the area perpendicular to the flow, the portion that is not occupied by soil particles. Therefore the actual velocity of the flow is larger than the apparent velocity v. It would be extremely difficult to determine the actual velocity of liquid molecules because of the tortuosity of the flow paths between soil particles. However, it is easy to determine the average velocity of the flow parallel to the direction of the flow, using the following relationship:

$$v_s = v/n \qquad \text{(Equation 2.2-3)}$$

where: $v_s$ = seepage velocity (i.e., average component of the actual velocity of liquid molecules parallel to the average direction of the flow); $v$ = apparent velocity; and $n$ = soil porosity.

Typical values of soil porosity are 0.25 - 0.50 (also expressed as percent: 25% - 50%).

The apparent velocity should be used when calculating the flow rate, while the seepage velocity should be used to calculate the time it takes a given liquid to flow from one point to another.

As indicated by Darcy's equation, the degree of permeability of a soil is expressed by its hydraulic conductivity, k. Typical values of soil hydraulic conductivities are given in Table 2.2-2.

Leakage rate through low-permeability compacted soils can significantly increase if the soil layer has defects such as those discussed in Section 2.2.2.4.3.

#### 2.2.2.3.3 Permeation through FMLs

The mechanism by which liquids migrate through FMLs is very complex and is discussed in Section 2.2.3.1.2.

FMLs are not porous media like soils and, therefore, flow of liquids or gases through FMLs is not governed by Darcy's equation. This is why the terminology "permeation through FMLs" is preferred to the terminology "permeability of FMLs". However, for the sake of comparison, permeability of FMLs can be expressed by an equivalent hydraulic conductivity. Typical values of equivalent hydraulic conductivities for FMLs range between $10^{-13}$ m/s and $10^{-16}$ m/s. These values show that permeation rates through FMLs are many orders of magnitude smaller than the flow rate through clays which are the least permeable soils. Leakage rates due to permeation through FMLs are usually very small compared to leakage rates due to FML defects (which are discussed in the next section).

### 2.2.2.4 Typical Defects of Liner Materials

#### 2.2.2.4.1 Introduction

As indicated in Section 2.2.2.3, leakage rates through low permeability soils as well as FMLs can significantly increase if these materials have defects.

There are several possible causes of defects in the FML component and low-permeability soil component of top liners that could lead to leakage through the top liner. Typical defects are discussed in the following two sections.

#### 2.2.2.4.2 FML Defects

Defects that are likely to occur to FMLs are numerous and may be caused by a wide variety of factors including improper design, defective manufacturing and defective installation. A number of publications are available which discuss various types of defects observed in FML-lined units [Bass, 1985,; Giroud, 1984a; Giroud, 1984c; Mitchell, 1984].

Typical defects observed in FMLs include:

- defective seams during fabrication, installation, or unit operation as a result of factors including excessive moisture or humidity, improper temperature, contamination by dust or dirt, undone seams, or excessive stresses during unit operation caused by improper design;
- damage to the FML during construction or facility operation as a result of excessive stresses caused by equipment;
- puncturing of FML by stones in the support or cover material;
- tensile failure of the FML due to excessive stresses generated by weight of stored material and movements of materials in contact with the lining system;
- fatigue failures caused by slow crack growth mechanisms prompted by repeated stresses such as thermal expansion-contraction; and
- inadequate connections between FMLs and appurtenances.

The last type of defect is illustrated by the following case history presented in Section 2.5.3. After the construction of a lining system for a landfill, and before placement of the waste, the lower portion of the landfill was filled with water to test the lining system. A leakage of 1000 liters/day (250 gallons/day) was observed at the connection between the FML and the sump, with a head of only

0.15 m (6 in.) of water above the defective connection.

The potential for the above mentioned defects to occur is minimal in properly designed and constructed FML-lined units. However, even in properly designed and constructed units, there is no guarantee that these defects will not occur. A discussion of the frequency and size of FML defects is presented in Section 2.2.3.2.

### 2.2.2.4.3 Low-Permeability Compacted Soils

The types of defects likely to occur with low-permeability compacted soil liners are also numerous. GCA [1986] summarized some of the potential types of defects associated with the construction of low-permeability compacted soil liners which may increase their hydraulic conductivity. These are reported below:

- Precipitation or other access to water may decrease the strength and increase the hydraulic conductivity of the uppermost lifts of low-permeability soil liners;

- Natural weathering (e.g., desiccation) can result in the formation of macrofeatures which increase the hydraulic conductivity of low-permeability soil liners;

- Occasional pockets of permeable material (e.g., sand lenses or silt) may be incorporated into lifts;

- Variability in the water content, compactive effort and clod size of the low-permeability soil can give rise to zones of higher hydraulic conductivity;

- There are likely to be variations in lift thickness during construction and nonuniformities at penetrations in the bottom and sidewall; and

- Insufficient bonding between adjacent lifts creates zones of high horizontal permeability.

Defects in a low-permeability soil layer increase its hydraulic conductivity, thereby increasing leakage rate through the top liner.

### 2.2.2.4.4 Composite Liner Defects

The preceding two sections address defects specific to each of the two components of a composite liner: the FML and the low-permeability soil. In addition to these defects, there are defects that are inherent to the composite liner itself, such as a poor contact between the FML and the low-permeability compacted soil. Poor contact may result from wrinkles in the FML and/or irregularities or clods at the surface of the low-permeability soil. FML wrinkles may exist even under very high pressures as shown by Stone [1984].

In addition, it should be noted that the placement of the low-permeability compacted soil of a composite top liner may damage the underlying geosynthetics, e.g., the FML of the composite bottom liner. This is especially true if the leak detection system is comprised of a geosynthetic material such as a geonet. Because synthetic drainage layers are thin and are not able to provide protection to the underlying FML component of a composite bottom liner, compaction of the bottom part of the low-permeability soil layer of composite top liners must be carried out with care. A frequent solution consists of placing a thicker first lift (e.g. 0.3 to 0.45 m (12 to 18 in.) in thickness) and compacting with heavy, rubber-tired equipment. However, with such thick lifts, the potential exists for not compacting sufficiently the bottom part of the lift adjacent to the leak detection drainage layer. As a result, part of this first lift probably has a higher hydraulic conductivity than the overlying, better compacted thin lifts.

## 2.2.3 Leakage through FML Top Liner

### 2.2.3.1 Introduction

#### 2.2.3.1.1 Scope of the Section

As indicated in Section 2.2.2.1, top liners can be either FMLs alone or composite liners. Section 2.2.3 discusses leakage through FML top liners. Leakage through composite top liners will be discussed in Section 2.2.4.

In Section 2.2.2.3.3, it was pointed out that leakage through a FML is very complex and can occur as a result of: (i) permeation through an intact FML; and (ii) flow through defects in a FML. Accordingly, Section 2.2.3 will discuss first leakage due to permeation through FMLs and, then, leakage due to FML defects. It should be understood that permeation through FMLs is an area of active research and that new information will become available in the near future. EPA will be soliciting new information and comments on FML permeation during the public comment period for the proposed Liner/Leak Detection Rule and this document will be updated to reflect the new information once it is available.

#### 2.2.3.1.2 Organization of the Section

The first part of Section 2.2.3 (i.e., Section 2.2.3.2) is devoted to leakage evaluation:

- Evaluation of leakage due to permeation through FMLs is addressed in Section 2.2.3.2.2.

- Evaluation of leakage due to flow through FML defects is addressed in Section 2.2.3.2.3 (pinholes) and 2.2.3.2.4 (holes).

This leakage evaluation indicates that leakage due to permeation is generally negligible as compared to leakage due to defects. Therefore, to determine leakage through a FML top liner it is necessary to estimate the number and size of defects in the FML. Accordingly the second part of Section 2.2.3 (i.e., Section 2.2.3.3) is devoted to an assessment of the frequency and size of FML defects using data from field experience.

Finally, conclusions regarding leakage through FML top liners are presented in Section 2.2.3.4.

### 2.2.3.2 Evaluation of Leakage through FML Top Liners

#### 2.2.3.2.1 Introduction

Three mechanisms of leakage are considered:

- leakage due to permeation through an intact FML;
- leakage through pinholes in the FML; and
- leakage through holes in the FML.

#### 2.2.3.2.2 Leakage due to Permeation through FML

- Permeameter Tests

Tests conducted at the University of Grenoble (France) by Giroud from 1973 to 1978 and, then, by Gourc and Faure, using a permeameter similar to those used to measure clay permeability (Figure 2.2-1), have shown that water permeates a FML.

Results of these tests have been published by Giroud [1984a, 1984c]. In these publications, Darcy's equation has been used to interpret the test results and calculate equivalent hydraulic conductivities which vary significantly with the hydraulic head (and, consequently, the hydraulic gradient).

- The Concept of Coefficient of Migration

It is preferable to interpret the permeameter tests discussed above using the following equation proposed by Giroud et al. [1987]:

$$Q/A = \mu_g/T \qquad \text{(Equation 2.2-4)}$$

where: Q = flow rate due to permeation through the FML; A = surface area of the considered FML; Q/A = flow rate per unit area; $\mu_g$ = coefficient of migration of the FML; and T = FML thickness. Recommended SI units are: Q ($m^3/s$), A ($m^2$), Q/A (m/s), $\mu_g$ ($m^2/s$), and T (m).

Values of the coefficient of migration for various FMLs are given in Table 2.2-3. Although there are not enough test results to draw a firm conclusion, it appears that the coefficient of migration increases as the hydraulic head increases to some maximum value, $\mu_{max}$. For hydraulic heads larger than approximately 10 meters (30 ft), $\mu = \mu_{max}$. The value of $\mu_{max}$ depends on the polymer used to make the FML. The value of $\mu$ is obviously zero for a hydraulic head equal to zero. Therefore, the typical shape of the curve of the coefficient of migration versus hydraulic head is given as shown in Figure 2.2-2.

It is difficult to conduct water permeability tests on FMLs with a head of water smaller than 5 m (16 ft) because the flow rates are too small to be accurately measured. The hydraulic heads that are relevant to hazardous waste land disposal units are usually smaller than 5 m (16 ft). Therefore it is useful to complement results from the permeameter tests cited above with results from water vapor transmission tests which are typically conducted with a pressure on the order of 1 to 10 kPa (0.15 to 1.5 psi), i.e., a hydraulic head on the order of 0.1 m to 1 m (4 in. to 40 in.).

- Water Vapor Transmission Tests

Water vapor transmission tests are typically performed on thin membrane materials because the mechanism for fluid transport through membranes is believed to be one of molecular diffusion through a nonporous membrane [Haxo et al., 1984]. With this mechanism, transport through the membrane involves three steps: (i) dissolution of the fluid into the membrane; (ii) diffusion of the fluid through the membrane; and (iii) evaporation or dissolution of the fluid on the downstream side of the membrane. According to Haxo et al. [1984], the major driving force for the movement of a given fluid through a membrane is its concentration gradient across the membrane. In the case of water, the important concentration gradient is suggested to be the water vapor pressure, and moisture is thought to move through the membrane by water vapor diffusion. It is important to note that water vapor diffusion decreases when the thickness of the membrane increases, but is not dependent on the hydraulic head acting on the membrane.

Haxo et al. [1984] have described a water vapor transmission test (ASTM E96, Procedure BW) and have used it to measure water vapor transmission rates for the range of FML materials given in Table 2.2-4. Values of water vapor transmission rates obtained from other sources are given in Table 2.2-5.

Knowing the water vapor transmission rate of a given FML obtained in a given test, the quantity of vapor permeating through this FML can be calculated using Fick's equation:

$$M/(At) = (WVT)\ (T_0/T)\ (\Delta p/\Delta p_0) \qquad \text{(Equation 2.2-5)}$$

where: $M$ = mass of vapor migrating through the FML; $A$ = FML surface area; $t$ = time (i.e., duration of the permeation); WVT = water vapor transmission rate; $T_0$ = FML thickness used in the water vapor transmission test; $T$ = considered FML thickness; $\Delta p$ = vapor pressure difference between the two sides of the considered FML; and $\Delta p_0$ = vapor pressure difference between the two sides of the FML used in the water vapor transmission test. Recommended SI units are: $M$ (kg), $A$ ($m^2$), $t$ (s), WVT ($kg/(m^2.s)$), $T_0$ and $T$ (m), and $\Delta p$ and $\Delta p_0$ ($N/m^2$). (Note: $1\ g/(m^2.day) = 1.16 \times 10^{-8}\ kg/(m^2.s)$).

Vapor pressure is given by:

$$p = p_s H \qquad \text{(Equation 2.2-6)}$$

where: $p_s$ = vapor pressure at saturated point; and H = relative humidity.

Therefore, Equation 2.2-5 can be written as follows:

$$M/(At) = (WVT)\ (T_o/T)\ (\Delta H/\Delta H_o) \qquad \text{(Equation 2.2-7)}$$

where: $\Delta H$ = relative humidity difference between the two sides of the considered FML; $\Delta H_o$ = relative humidity difference between the two sides of the FML used in the water vapor transmission test; and other notation as for Equation 2.2-5.

It should be pointed out that the use of Equations 2.2-5 and 2.2-7 should be restricted to pressures that are not too different from the pressures typically used to conduct water vapor transmission test (e.g., pressures on the order of 1,000 to 10,000 Pa (0.15 to 1.5 psi), i.e., hydraulic heads on the order of 0.1 to 1 m (4 to 40 in.) of water).

According to Fick's equation (Equation 2.2-5), there is no permeation through an FML if the relative humidity is the same on both sides of the FML. This is the case, in particular, if there is water on both sides, even if there is a pressure difference. This is in disagreement with results obtained using a permeameter, which were presented at the beginning of Section 2.2.3.2.2. More research is therefore needed on this subject.

- Relationships between Various Expressions of Flow Rate

In order to use water vapor transmission test results to complement permeameter test results, it is necessary to establish relationships between the various coefficients used to express flow rate.

An equivalent hydraulic conductivity for FMLs can be obtained by expressing flow rate through a FML using Darcy's equation:

$$v = Q/A = k_g\ i \qquad \text{(Equation 2.2-8)}$$

where: $v$ = apparent velocity of the flow; $Q$ = flow rate; $A$ = area perpendicular to the flow; $k_g$ = equivalent hydraulic conductivity of the FML; and $i$ = hydraulic gradient.

By comparing Equation 2.2-4 with Equation 2.2-8, it appears that:

$$\mu_g = k_g\ h \qquad \text{(Equation 2.2-9)}$$

where: $\mu_g$ = coefficient of migration of the FML; $k_g$ = equivalent hydraulic conductivity of the FML; and $h$ = hydraulic head. Recommended SI units are: $\mu_g$ ($m^2/s$), $k_g$ (m/s), and $h$ (m).

By comparing Equation 2.2-8 (Darcy's equation) with Equation 2.2-5 (Fick's equation), it appears that:

$$WVT = p\ k_g/(g\ T) = \rho\ k_g\ h/T \qquad \text{(Equation 2.2-10)}$$

By combining Equations 2.2-9 and 2.2-10, it comes:

$$WVT = \rho\ \mu_g/T \qquad \text{(Equation 2.2-11)}$$

where: $k_g$ = FML equivalent hydraulic conductivity; $g$ = acceleration of gravity; $T$ = FML thickness; WVT = FML water vapor transmission rate; $p$ = pressure; $\rho$ = liquid density; $h$ = hydraulic head; and $\mu_g$ = coefficient of migration. The recommended SI units are: $k_g$ (m/s), $g$ ($m/s^2$), $T$ (m), WVT ($kg/(m^2.s)$), $p$ (Pa), $\rho$ ($kg/m^3$), $h$ (m), and $\mu_g$ ($m^2/s$). A useful conversion factor for WVT is:

$$1g/(m^2.24h) = 1.16 \times 10^{-8}\ kg/(m^2.s)$$

Using Equation 2.2-11, the measured water vapor transmission (WVT) values given in Tables 2.2-4 and 2.2-5 have been converted into values of the coefficient of migration. It is interesting to see in Table 2.2-4 that series of tests on a given product (e.g., series of four tests on PVC) with various thicknesses generally give consistent values of the coefficient of migration.

There are not enough values in Tables 2.2-3, 2.2-4 and 2.2-5 to establish a complete table of values of coefficient of migration, $\mu_g$, for FMLs. It is therefore necessary to draw curves such as those in Figure 2.2-3 to make interpolations and extrapolations for small values of the hydraulic head. Also, Tables 2.2-3, 2.2-4 and 2.2-5 contain discrepancies and apparently erratic results due to the difficulty of the tests and the sometimes great differences between FMLs of the same type. Therefore some averaging was necessary. Values of the coefficient of migration from Tables 2.2-3, 2.2-4 and 2.2-5 are summarized in Table 2.2-6. Figure 2.2-3 was established using values of the coefficient of migration given in Table 2.2-6.

The large discrepancy between water vapor transmission rates measured on PVC at 0.14 m head (Table 2.2-4) and 0.6 m head (Table 2.2-5) probably results from the fact that the PVC tested at 0.14 m head was a FML made of plasticized PVC and the PVC tested at 0.6 m was pure PVC. Plasticized PVC is swelled by the plasticizers and tends to be more permeable than pure PVC (such as the stiff PVC used to make bottles, which has a very low permeability).

- Leakage Rate Evaluation

From Figure 2.2-3, it is possible to establish Table 2.2-7 which gives our best estimate of coefficient of migration values from the analyzed data. From Table 2.2-7, it is possible, using Equation 2.2-4, to establish Table 2.2-8 which gives leakage rates due to permeation through FMLs, assuming an FML thickness of 1 mm (40 mils).

- Migration of Chemicals

Many types of FMLs swell when placed in contact with chemicals. As a result, the distance between polymeric chains increases and permeability increases. Therefore, an FML can have a low permeability to water and a high permeability to some chemicals. Data regarding permeation of FMLs by chemicals can be found in [Haxo et al., 1984, 1986] and [Telles et al., 1986].

### 2.2.3.2.3 Leakage due to Pinholes in the FML

- Definition of Pinholes

According to Giroud [1984b]: pinholes should be distinguished from holes and can be defined as openings having a dimension (such as diameter) significantly smaller than the FML thickness. The primary source of pinholes are manufacturing defects. Early manufacturing techniques for FMLs often resulted in a significant number of FML defects. However, manufacturing processes and polymer formulations have advanced to a degree that pinholes are now relatively rare. Good resin selection and manufacturing quality control should virtually eliminate pinholes. This discussion of pinholes is included herein for completeness. Leakage through FML pinholes will not, however, be included in the final calculations of total leakage through FML top liners.

- Basic Equation

For leakage calculation purposes, pinholes can be considered as pipes and, therefore, according to Giroud [1984b], Poiseuille's equation can be used:

$$Q = \pi \rho g h d^4/(128 \eta T) \qquad \text{(Equation 2.2-12)}$$

where: $Q$ = leakage rate; $h$ = hydraulic head on top of the FML; $T$ = thickness of the FML; $d$ = pinhole diameter; $\rho$ and $\eta$ = density and dynamic viscosity of leachate; and $g$ = acceleration of gravity. Recommended SI units are: $Q$ ($m^3/s$), $h$ (m), $T$ (m), $d$ (m), $\rho$ ($kg/m^3$), $\eta$ (kg/m.s), and $g$ ($m/s^2$).

The above equation is different from the equation used for evaluating leakage through holes (see Equation 2.2-12, Section 2.2.3.2.4).

- Calculations

Equation 2.2-12 has been used to calculate leakage rates for two typical pinhole diameters, 0.1 mm (0.004 in.) and 0.3 mm (0.012 in.), assuming a FML thickness of 1 mm (40 mils). Results are in Table 2.2-9. The hydraulic heads used in these calculations are as follows:

- 0.03 m (0.1 ft) which is an average head that can normally be expected on the top liner of a landfill with a well designed and constructed leachate collection and removal system;
- 0.3 m (1 ft) which is the maximum head considered in the design of the leachate collection and removal system of a landfill; and
- 3 m (10 ft) which is a typical head on the top liner of a surface impoundment.

### 2.2.3.2.4 Leakage due to Holes in the FML

- Definition of Holes

According to Giroud [1984b] holes should be distinguished from pinholes and can be defined as openings having a dimension (e.g., diameter) about as large as, or larger larger than, FML thickness.

- Assumption Regarding Underlying Material

Leakage rates through FML holes are significantly affected by the nature of the material underlying the FML. Two extreme cases can be considered: a high-permeability material such as a granular or synthetic drainage medium, and a low-permeability compacted soil such as a clay layer placed under an FML to form a composite liner. The case of a composite liner is addressed in Section 2.2.4.

In this section, the material underlying the FML is assumed to have an infinite hydraulic conductivity. Tests by Brown et al. [no date] have shown that underlying soils with a hydraulic conductivity higher than $10^{-3}$ m/s ($10^{-1}$ cm/s) do not affect free flow through a FML defect, which justifies the assumption of an infinite hydraulic conductivity for drainage materials supporting a FML top liner.

- Assumption Regarding Overlying Material

Leakage rates through FML holes are affected by the material overlying the FML. The more permeable this material, the higher the leakage rate will be. Therefore, in subsequent calculations, the overlying material will conservatively be assumed to be infinitely pervious.

Since it is possible that the hydraulic conductivity of the soil overlying the FML has a marked influence on leakage rates through top liners, especially in the case of surface impoundments where hydraulic heads are high, research should be done on this topic.

- Basic Equation

Assuming that the considered FML is located between two infinitely pervious media, Bernouilli's equation for free flow through orifices can be used to evaluate the leakage rate through a hole in the FML:

$$Q = C\,a\,\sqrt{2gh} \qquad \text{(Equation 2.2-13)}$$

where: Q = leakage rate; h = hydraulic head on top of the FML; a = hole surface area; and g = acceleration of gravity. C is a dimensionless coefficient, valid for any Newtonian fluid, and is related to the shape of the edges of the aperture; for sharp edges, C = 0.6. Recommended SI units are: Q ($m^3/s$), h (m), a ($m^2$), and g ($m/s^2$).

- Calculations

Equation 2.2-13 has been used to calculate leakage rates for two typical holes:

- a 2 mm (0.08 in.) diameter hole which is assumed to be a "small" hole due to defective seaming (as discussed in Section 2.2.3.3.5) and might escape detection by construction quality assurance; and

- a 11.3 mm (0.445 in.) diameter hole which is a "standard" 1 cm² hole conservatively recommended for design, as indicated in Section 2.2.3.3.6.

Results from these calculations are given in Table 2.2-9. Hydraulic heads considered in these calculations are as follows:

- 0.03 m (0.1 ft) which is assumed to be the average head acting on the top liner of a landfill with a well designed and constructed leachate collection and removal system.

- 0.3 m (1 ft) which is the maximum head considered in the design of the leachate collection and removal system of a landfill.

- 3 m (10 ft) which is assumed to be the maximum head on the top liner of a surface impoundment.

### 2.2.3.3 Frequency and Size of FML Defects

#### 2.2.3.3.1 Purpose

The purpose of this section is to evaluate the size and frequency of defects which can occur in a FML. (Causes of defects were discussed in Section 2.2.2.3.) This information is necessary for making analytic calculations to evaluate leakage through top liners (FML alone as well as composite liners). Although this section is devoted to all types of defects, it focuses primarily on seam defects because forensic analyses have shown that leakage through FML liners is often due to defective seams, and the most complete documentation of FML defects is for seam defects.

This section is organized as follows: first, data from construction quality assurance and forensic analyses are reviewed, then conclusions are drawn from these data.

### 2.2.3.3.2 Data from Construction Quality Assurance

- Small Liquid Reservoir

This project, constructed in 1981, is described in detail by Giroud and Stone [1984], and Stone [1984]. Information regarding seam defects can be summarized as follows.

The double liner system includes two 2.5 mm (100 mil) thick HDPE FMLs which were welded using an automated extrusion welder. Ultrasonic testing, carried out as part of the quality control and quality assurance program, showed that approximately 0.5% of the seam length was defective. The detected defects were repaired and the reservoir was filled with water. Leakage occurred and an inspection showed that leakage was taking place through approximately 0.015% of the seam length. The ratio 0.5/0.015 shows that, in this project, intensive quality assurance divided the length of defects by approximately 30.

This project is particularly interesting because it provides an evaluation of the benefits from construction quality assurance.

- Large Landfill with Single Liner

Kastman [1984] indicates that in a carefully monitored landfill liner installation done in 1983, approximately one defect every 15 m (50 ft) of seam was detected and repaired, as part of the quality assurance process. The liner was a 1 mm (40 mil) thick HDPE FML and seaming was achieved with a fillet extrusion weld done using a hand welder.

- Large Landfill with Double Liner

Giroud and Fluet [1986] report the result of an analysis conducted on the basis of data collected during the quality assurance process of liner installation in a large landfill, lined in 1985 with an HDPE FML. The surface area of the liner is approximately 35 000 m$^2$ (350,000 ft$^2$) and seam length is approximately 5000 m (16,000 ft). During the quality assurance process, an average of approximately one seam defect every 9 m (30 ft) of seam length was discovered and repaired.

- Large Landfill with Single Liner

This case history presents the results of an analysis conducted on the basis of data available in GeoServices files. The data were collected during the installation of the lining system in a large landfill, in 1987, as part of the quality control provided by the FML installer and quality assurance provided by an independent firm. The surface area of the liner is approximately 53 000 m$^2$ (570,000 ft$^2$) and seam length is approximately 8000 m (26,000 ft). The liner was a 1.5 mm (60 mil) thick HDPE FML. Half of the seam length was welded using a hand welder which made fillet extrusion welds; the other half was welded using an automated flat welder. An average of approximately one seam defect every 11.5 m (38 ft) of seam length was discovered by the FML installer and the independent quality assurance firm. All these defects were repaired. Seam inspection was performed first by the installer, and then by the independent firm after the installer had completed his inspection. The installer detected approximately one seam defect every 17 m (56 ft) of seam length. The independent firm detected approximately one seam defect every 35 m (115 ft) of seam length.

This project is interesting because it provides an evaluation of the benefits from construction quality assurance. The independent firm discovered additional seam defects, after the installer had completed his quality control inspection. The defects discovered by the independent firm totaled one third of the total seam defects. The benefits of quality assurance are probably greater than that: it is probable that, without the continuous presence at the site of the independent quality assurance firm, the FML installer would have found fewer defects than he did as part of his quality control effort.

### 2.2.3.3.3 Data from Forensic Analyses

- Small Indoor Tank

A power generating station required a small acid holding tank, which was constructed of concrete and lined in 1985 with a high density polyethylene (HDPE) FML which required approximately 100 m (300 ft) of field seaming. The seams were fillet welds done with a hand welder. The design and installation included no third party quality assurance, but careful quality control of seaming was provided by the installer, using visual inspection and vacuum box.

Upon completion of the liner installation, the tank was filled with water to check for leaks. The liner did leak, so the tank was emptied, repairs were made and the tank was filled again. This cycle was repeated several times, with leaks found on every filling. Leaks were found at 15 different locations, i.e. an average of one leak per 7 m (23 ft) of seam. This incidence of seam defects is probably significantly larger than the incidence which would be experienced in FML installations in land disposal units, given the complex geometry and difficult welding conditions in the holding tank compared to a land disposal unit. However, the complex geometry of this tank is probably representative of the difficulties encountered in land disposal units at the connections between FMLs and appurtenances such as sumps.

- Large Surface Impoundment

The following case history is reported by Giroud and Fluet [1986].

A large reservoir, lined with a single reinforced chlorosulfonated polyethylene (CSPE-R) FML, had been constructed to contain phosphoric

One year after the first filling, the reservoir suddenly emptied. The analysis of the failure indicated that phosphoric acid, leaking through several defective seams, attacked the ground, creating cavities. The largest cavity was one meter (three feet) in diameter and half a meter (20 inches) deep. Under the pressure of the impounded liquid, the FML spanning this largest cavity burst, releasing all of the impounded phosphoric acid into the ground.

Quality assurance during installation had consisted of only two one-day visits by an engineer who specialized in roofing membranes. Therefore, it is not surprising that defective seams were not detected prior to filling.

During the forensic analysis, visual observation showed that approximately 0.1% of the seam length was defective. It is probable that a higher percentage would have been obtained if a vacuum box had been used instead of the visual inspection.

#### 2.2.3.3.4 Conclusions on Frequency of Defects

- Consistency of the Observations

Sections 2.2.2.2 and 2.2.2.3 present data related to frequency of seam defects. Some of these data are expressed as an average seam length exhibiting one defect (e.g., one defect per 7 m (23 ft) of seam), while other data are expressed as percentage of defective seam length (e.g., 0.5% of the total seam length was defective).

If an average length of seam defect (prior to quality assurance) of 10 mm (0.4 in.) is considered, a percentage of defective seam length of 0.1% is equivalent to one defect every 10 m (30 ft). Therefore, the observations from the previous case studies appear to be consistent.

- Conclusion Regarding Frequency of Seam Defects

It is not possible to draw general conclusions from only six cases. However, since the observations made in these six cases were consistent it is possible to draw the following tentative conclusions:

- An average of one defect per 10 m (30 ft) of seam can be expected without quality assurance.

- An average of one defect per 300 m (1,000 ft) of seam can be expected with reasonably good installation, adequate quality assurance, and repair of noted defects. Even better results (possibly up to one defect per 1000 m (3000 ft)) may be possible with the very best installation procedures and intensive quality assurance. (Quality assurance followed by repair drastically decreases the number of seam defects but may not totally eliminate them.)

The average of one seam defect per 10 m (30 ft) without or before quality assurance will probably decrease in the future as a result of the increasing use of new, automated methods of seaming which are now available. However, the number of seam defects after quality assurance may not decrease significantly because, in the present state of practice for construction quality assurance, great emphasis is put on finding seam defects and repairing them. Nonetheless, the better seaming methods that are now available are highly beneficial for at least the following reasons: (i) less seam repair is required during installation; (ii) frequency of destructive seam testing may be decreased; (iii) quality assurance efforts may shift toward other areas where improvement is sorely needed such as connections of FMLs with appurtenances and placement of drainage materials (which is essential for the functioning of leak detection systems); and (iv) stronger seams that are less likely to fail when subjected to stresses.

As a result of the above discussion, a frequency of one defect per 300 m (1,000 ft) of seam will be used as a conservative working assumption. If FML panels 6 to 10 m (20 to 30 ft) wide are used, one defect per 300 m (1,000 ft) of seam is equivalent to 3 to 5 seam defects per hectare (1 to 2 seam defects per acre) of installed FML.

As soon as possible, these tentative conclusions must be supplanted and modified as required by conclusions established on a broader base of well documented case histories. In the meantime (and in the absence of better data), an arbitrary frequency of one defect per 4000 $m^2$ (acre) will be used in calculations for estimating leakage rates in order to size leak detection drainage layers. Such frequency is assumed to include all types of defects, not only seam defects.

### 2.2.3.3.5 Estimation of Size of Defects

The seam defect documentation reported above addressed primarily the frequency of seam defects. Extensive documentation of defect size does not exist. On the basis of interviews with quality assurance personnel it appears that the maximum size of defects which may still exist after intensive quality assurance is equivalent to hole diameters on the order of 1 to 3 mm (0.04 to 0.12 in.) for seam defects and possibly up to 5 mm (0.2 in.) for special areas such as connections of FML with appurtenances. This is confirmed by the case history presented in Section 2.5.3.

There are also defects that cannot be observed by the quality assurance personnel, such as: (i) puncture of the FML during installation of the protective earth cover; and (ii) puncture of the FML as a result of stresses due to the weight of waste or traffic related to the operation of the hazardous waste management unit. Therefore, for design purposes it may be appropriate and conservative for subsequent calculations to consider a hole larger than the expected size of defects at the end of FML installation (which were estimated above as 5 mm (0.2 in.) maximum in diameter). However, for the establishment of the best demonstrated available technology (BDAT) for liner installations, a smaller hole size (in the range of 1 to 2 mm (0.04 to 0.08 in.)) is probably more appropriate.

#### 2.2.3.3.6 Standard Hole Size and Frequency

For the consistency of calculations and discussions supporting the proposed Liner/Leak Detection Rule, it is recommended that a standard hole size and frequency be selected. The same standard hole size and frequency will also be useful as guidance for designers of leak detection systems.

As a result of the above discussions, a "standard" hole area of 1 cm$^2$ ($10^{-4}$ m$^2$ or 0.16 in$^2$.) has conservatively been selected, and, on the basis of the discussion presented in Section 2.2.3.3.4, a frequency of one standard hole per 4000 m$^2$ (acre) is considered. The standard hole area and frequency are used in this background document for calculations done to evaluate the leakage rates used to establish the required drainage capabilities of the LDCRS.

It should be kept in mind that the standard hole size and frequency have been selected with the assumption that good quality assurance monitoring will be performed. Also, the standard hole size and frequency does not take into account cases where design flaws or poor construction practices would lead to many seam defects or a large tear in the FML.

Lastly, as previously noted, the standard hole size is for design and calculation purposes and not for the purpose of defining BDAT. Selection of a standard hole size for design should include a margin of safety. Selection of a hole size for BDAT should be based on the best actual installation and construction quality assurance practice. On this basis, the small hole size is probably more representative of BDAT than the standard hole size.

### 2.2.3.4 Conclusions on Leakage through FML Top Liners

#### 2.2.3.4.1 Summary

In Section 2.2.3, the following was done. The results of permeameter tests and water vapor transmission tests were used to evaluate permeation through FMLs. Equations to evaluate leakage rate through pinholes and holes were presented. The following recommendation regarding frequency and size of holes to be considered in calculations was made on the basis of field experience: one 1 $cm^2$ (0.16 $in^2$.) hole per 4000 $m^2$ (acre).

#### 2.2.3.4.2 Leakage Rates

By combining Tables 2.2-8 and 2.2-9, it is possible to establish Table 2.2-10 which gives orders of magnitude of typical leakage rates which can be expected when a FML is used alone as a top liner.

It appears that the leakage rate due to only one hole per 4000 $m^2$ (acre) is very large, while the leakage rate due to permeation and pinholes is small. It must be remembered that the leakage rates given in Table 2.2-10 are related to a top liner comprised of a FML placed directly on the leak detection, collection, and removal system. As indicated in Section 2.2.3.2.4, the equation used is valid if the hydraulic conductivity of the leak detection, collection, and removal system is larger than $10^{-3}$ m/s (0.1 cm/s), which should always be the case.

### 2.2.4 Leakage Through Composite Liners

### 2.2.4.1 Introduction

#### 2.2.4.1.1 Purpose of the Section

This section discusses leakage through composite liners due to a hole in the FML. The purpose of this discussion is to draw practical conclusions regarding the evaluation of leakage rate through composite top liners. These conclusions will be used in the calculations made in subsequent sections to compare top liners comprised of a FML alone with composite top liners, and to evaluate the rate of leakage that impinges into the leak detection, collection, and removal system. Composite top liners are not now required by EPA. However, some owners and operators are opting for composite top liners because, as shown in this section, they reduce leakage through the top liner in comparison to the leakage through a FML alone. However, as will be shown in Section 2.10, the use of a top composite liner has one drawback from the standpoint of the proposed Liner/Leak Detection Rule: consolidation of the compacted soil component of the top composite liner will result in a significant flow of "consolidation water" into the LDCRS.

### 2.2.4.1.2 Leakage Mechanisms

A composite liner is comprised of a FML (which is the upper component of the composite liner) and a low-permeability compacted soil layer (which is the lower component of the composite liner). If there is leakage through a composite liner, the leachate first migrates through the FML, then may travel laterally in the space, if any, between the FML and the low-permeability compacted soil, and, finally, migrates through the low permeability soil.

There are three mechanisms by which leakage can migrate through a FML:

- permeation through the FML (i.e., flow through a FML that has no defects such as holes or pinholes);
- flow through pinholes in the FML (pinholes are very small openings that have a diameter that is significantly smaller than the thickness of the FML); and
- flow through holes in the FML (holes are openings that have a diameter larger than the thickness of the FML).

Leakage rate due to permeation through the FML should not be significantly affected by the presence of the low-permeability compacted soil layer under the FML because even a soil with a very low permeability is still very permeable as compared to a FML without holes and pinholes. The case of permeation through a FML without holes or pinholes was discussed in Section 2.2.3.2.2.

Section 2.2.4 is devoted to leakage through composite liners due to a defect such as pinhole or hole in the FML. However, pinholes are only briefly discussed since pinholes are rare and obviously cause much less leakage than a hole.

#### 2.2.4.1.3 Organization of the Section

The remaining three subsections are devoted to:

- analytical studies;
- laboratory models; and
- practical conclusions regarding the evaluation of leakage rate through composite liners.

### 2.2.4.2 Analytical Studies

#### 2.2.4.2.1 Introduction

As indicated in Section 2.2.4.1.2, the leachate that has passed through the FML can flow laterally to a certain extent between the FML and the low-permeability compacted soil, before it migrates through the low permeability soil. This is possible if there is a space between the FML and the low permeability soil.

Two types of analytical studies can be found in the literature:

- analytical studies assuming that there is perfect contact between the FML and the low-permeability compacted soil, and, consequently, that the leachate does not flow laterally between the FML and the low-permeability compacted soil; and
- analytical studies assuming that leachate flows laterally between the FML and the low-permeability compacted soil before it migrates through the low permeability soil.

### 2.2.4.2.2 Analyses Assuming Perfect Contact

- Assumptions

Faure [1979] has made an extensive study of the leakage rate through a composite liner due to a hole in the FML, assuming perfect contact between the FML and the underlying low permeability soil. First, Faure considered two simple two-dimensional cases:

- flow net established by considering that the entire soil layer is saturated (Figure 2.2-4 a); and
- radial flow (Figure 2.2-4 b) which leads to a convenient close form solution for the leakage rate (the radial flow was thought to be a reasonable assumption for thick soil layers, but in fact is not, as shown by Faure (see Figure 2.2-7)).

These two types of flow lead to absurd results (such as flow rate increasing when soil thickness increases). However, those cases are useful because Faure showed that they provide upper boundaries for the actual flow rate through the composite liner when the FML and the underlying soil are in perfect contact. Also the leakage rate in the case of the radial flow is expressed by a close form solution for the three-dimensional case (circular hole), which provides a convenient upper boundary for the three-dimensional case. This is very useful because the three-dimensional case is very difficult to analyze and this upper boundary is one of the few theoretical data available for the three-dimensional case.

A lower boundary of the leakage rate is obtained by assuming that the flow is vertical (Figure 2.2-4 c).

The actual flow if the FML and the low-permeability compacted soil are in perfect contact is shown in Figure 2.2-4 d. This has been demonstrated in the two-dimensional case by:

- Faure [1979] who used numerical methods; and

- Sherard [1985] who traced flow nets by trial and error.

Both Faure and Sherard have shown that, in a two-dimensional flow:

- there is horizontal flow in the soil along a portion of the interface (although there is no flow between the FML and the soil because there is no space between the FML and the soil when perfect contact is assumed); and

- there is a phreatic surface beyond which the soil is not saturated.

These qualitative characteristics of the flow are certainly also applicable to the three-dimensional case (circular hole). Typical flow nets for the two-dimensional case are given in Figure 2.2-5 and a chart giving the location of the phreatic surface in the two-dimensional case is presented in Figure 2.2-6.

- Leakage Rates for the Two-dimensional Case

Leakage rates obtained with the various assumptions discussed above are given in Figure 2.2-7 adapted from Faure. This figure shows that:

- absurd results are obtained with the upper boundaries, cases (a) and (b), when the low-permeability compacted soil thickness, H, is large; and

- case (c) is a very low lower boundary when the low-permeability compacted soil thickness, H, is large.

A chart giving the actual leakage rate (i.e., the leakage rate obtained in case d) when the FML and the underlying soil are in perfect contact has been prepared by Faure [1979, 1984] for the two-dimensional case (Figure 2.2-8). The results given by Sherard [1985] for a limited number of cases are consistent with Faure's. Faure's chart (Figure 2.2-8) is used with the following equation:

$$Q/B = C\ k_c\ (H + h) \qquad \text{(Equation 2.2-14)}$$

where: Q = leakage rate; B = length of the slot in the direction perpendicular to the figure; Q/B = leakage rate per unit length; C = dimensionless coefficient given by the chart; $k_c$ = hydraulic conductivity of the low-permeability compacted soil underlying the FML; H = thickness of the low-permeability compacted soil; and h = hydraulic head on top of the FML.

The equation for the two-dimensional radial flow (case (b) in Figures 2.2-4 and 2.2-7) which gives an upper boundary for the actual leakage rate is obtained by integrating Darcy's equation for a circular domain:

$$Q/B = \pi\ k_c\ (h + H)/\text{Log}\ (2H/b) \qquad \text{(Equation 2.2-15)}$$

where: Q = leakage rate; B = length of the slot in the direction perpendicular to the figure; Q/B = leakage rate per unit length; $k_c$ = hydraulic conductivity of the low-permeability compacted soil; h = head on top of the FML; b = width of the slot; and H = thickness of the low permeability soil underlying the FML. Recommended SI units are : Q ($m^3/s$); Q/B ($m^3/s/m$, i.e., $m^2/s$); $k_c$ (m/s); h (m); b (m); and H (m).

The equation for the vertical flow (case (c) in Figures 2.2-4 and 2.2-7), which gives a lower boundary for the flow rate, is obtained by writing Darcy's equation for a rectangular domain:

$$Q/B = k_c\ b\ (h + H)/H \qquad \text{(Equation 2.2-16)}$$

where the notation is the same as above.

This lower boundary gives a good approximation of the actual leakage rate if the ratio between the width of the FML hole and the thickness of the low permeability soil is large, which is rare.

The upper boundary provided by the radial flow (Equation 2.2-15) is excessively high in many cases and increases when H/h is large and increases, as shown in Figure 2.2-5. Since the leakage rate cannot increase if the thickness of the soil layer increases, the upper boundary is increasingly far from the actual leakage rate when H/h increases and, therefore, cannot be used as an approximation for the actual leakage rate.

Equation 2.2-15 can be arbitrarily transformed by replacing h + H by h, which gives:

$$Q/B = \pi k_c h/\mathrm{Log}\,(2H/b) \qquad \text{(Equation 2.2-17)}$$

As it turns out, this equation can be used for large values of H/h where it gives a lower boundary (case ($b_2$) in Figure 2.2-7) of the actual leakage rate which is not too far from the actual value (case (d) in Figure 2.2-7).

These considerations regarding boundaries will be useful to provide guidance for an approximate evaluation of the leakage rate in the three-dimensional case (circular hole) where the actual value of the leakage rate is not known.

- Leakage Rate for the Three-Dimensional Case

In the case of a three-dimensional flow (circular hole), the actual flow is certainly limited by a bell-shaped phreatic surface similar to the phreatic surface of the two-dimensional flow (case (d) in Figure 2.2-4, and Figure 2.2-5). However, no analytical or numerical study is presently available to the best of our knowledge. An upper boundary and a lower boundary are available and they are expressed by close-form solutions.

The equation related to the three-dimensional radial flow (similar to the two-dimensional case (b) in Figure 2.2-7), which gives an upper boundary for the actual leakage rate, is obtained by integrating Darcy's equation for a spherical domain:

$$Q = \pi k_c (h + H) d/(1 - 0.5d/H) \qquad \text{(Equation 2.2-18)}$$

where: $Q$ = leakage rate; $k_c$ = hydraulic conductivity of the low-permeability compacted soil; $h$ = hydraulic head on top of the FML; $d$ = diameter of the circular hole; and $H$ = thickness of the low permeability soil. Recommended SI units are: $Q$ (m³/s), $k_c$ (m/s), $h$ (m), $d$ (m), and $H$ (m).

The equation related to the vertical flow (similar to the two-dimensional case (c) in Figure 2.2-4), which gives a lower boundary for the actual leakage rate, is obtained by writing Darcy's equation for a cylindrical domain:

$$Q = k_c a (h + H)/H \qquad \text{(Equation 2.2-19)}$$

where: $a$ = surface area of the hole in the FML ($a = \pi d^2/4$ if the hole is circular); and other notation as above.

As discussed for the two-dimensional case, Equation 2.2-18 can be rewritten as follows:

$$Q = \pi k_c h d/(1 - 0.5d/H) \qquad \text{(Equation 2.2-20)}$$

It is possible that this equation gives a lower boundary of the actual leakage rate when $d/H$ is small (like Equation 2.2-17 for the two-dimensional case). It is interesting to note that Equation 2.2-20 tends toward a very simple limit when $d/H$ tends toward zero:

$$Q = \pi k_c h d \qquad \text{(Equation 2.2-21)}$$

where: $Q$ = leakage rate; $k_c$ = hydraulic conductivity of the low-permeability compacted soil underlying the FML; $h$ = hydraulic head on top of the FML; and $d$ = diameter of the circular hole in the FML.

Due to the lack of any better solution, Equation 2.2-21 will be used as an approximation for the actual leakage rate.

Another approach for evaluating leakage rate in the three-dimensional case is to use the chart established by Faure for the two-dimensional case (Figure 2.2-8) and modify Equation 2.2-14 by replacing the length B of the slot by the perimeter $\pi d$ of the circular hole (and not half the perimeter, nor the diameter of the hole as one may be tempted to do):

$$Q = \pi C k_c (H + h) d \qquad \text{(Equation 2.2-22)}$$

where: $Q$ = leakage rate; $C$ = dimensionless coefficient given by Faure's chart (Figure 2.2-8); $k_c$ = hydraulic conductivity of the low-permeability compacted soil; $H$ = thickness of the low permeability soil layer; $h$ = hydraulic head on top of the FML; and $d$ = hole diameter.

#### 2.2.4.2.3 Analyses Assuming Flow between FML and Soil

- Introduction

Analytical studies have been conducted by Fukuoka [1986] and Brown et al. [no date]:

- Fukuoka considered the case where there is a geotextile (without a hole) between the FML (with a hole) and the soil. The liquid leaking through the FML hole first flows horizontally in the geotextile, then vertically through the soil layer.

- Brown et al. considered that there is a space between the FML and the soil layer. The liquid leaking through the FML hole first flows horizontally in the space, then vertically through the soil layer.

- Flow in Geotextile between FML and Soil

Fukuoka [1986] considered the case of a geotextile between the FML and the low-permeability soil, and assumed that the leachate flows horizontally and radially within the geotextile before it flows vertically in the soil underlying the geotextile. Although geotextiles are not used in composite liners, the analysis made by Fukuoka is pertinent to composite liners because equations similar to those derived by Fukuoka can be used for flow in the narrow space (if it exists) between a FML and soil.

The following differential equation has been established by Fukuoka [1986]:

$$(1/r)\ (dh/dr) + d^2h/dr^2 = h\ k_c/(H\theta) \qquad \text{(Equation 2.2-23)}$$

where: $r$ = radius from center of hole; $h$ = hydraulic head at radius $r$ in the geotextile; $k_c$ = hydraulic conductivity of the low-permeability compacted soil underlying the geotextile; $H$ = thickness of the soil layer; and $\theta$ = hydraulic transmissivity of the geotextile.

The only assumption is that the flow in soil is vertical. No assumption is made regarding the hydraulic head in the geotextile. This head decreases from a maximum value at the FML hole, to zero at the periphery of the wetted portion of the geotextile. Consequently, flow through soil is faster at the center of the wetted area than at the periphery. Solving the above equation would give the radius of the wetted area and would allow the leakage rate to be determined. Fukuoka did not solve the equation, but the solution proposed by Brown et al. for Equation 2.2-29, which is similar, can be adapted to Equation 2.2-23 if the thickness of the geotextile (and, therefore, its transmissivity) is assumed not to vary with the radius r (while, in fact it varies since the effective stress on the geotextile varies with the radius r).

Equation 2.2-23 was established by combining Darcy's vertical flow in the soil with Darcy's radial flow in the geotextile, $Q_r$, which is governed by the classical differential equation:

$$Q_r = -\ 2\ \pi\ r\ k_p\ s\ dh/dr \qquad \text{(Equation 2.2-24)}$$

where: $k_p$ = hydraulic conductivity of the geotextile in the direction of its plane; s = thickness of the geotextile (i.e., spacing between FML and soil); and other notation as above.

This equation can also be written:

$$Q_r = -\,2\,\pi\,r\,\theta\,dh/dr \qquad \text{(Equation 2.2-25)}$$

where: $\theta$ = hydraulic transmissivity of the geotextile.

- Flow in Space between FML and Soil

This study was made by Brown et al. principally to extrapolate results obtained with their small diameter model to real situations where the flow may laterally extend over a large area.

The approach used by Brown et al. is similar to Fukuoka's. They combine vertical Darcy's flow in the low-permeability compacted soil with radial flow in the space between the FML and the underlying soil. Brown et al. integrated Newton's equation for viscous fluids in a circular domain and demonstrated that the radial flow is governed by:

$$Q_r = -\,[\pi\,r\,s^3\,\rho\,g/(6\,\eta)]\,(dh/dr) \qquad \text{(Equation 2.2-26)}$$

where: r = radius from center of hole; s = spacing between FML and low permeability soil; $\rho$ = density of leachate; g = acceleration of gravity; $\eta$ = viscosity of leachate; and h = hydraulic head at radius r in the space between FML and soil.

By comparing Equations 2.2-25 and 2.2-26, it appears that a space s between the FML and the underlying soil is equivalent to a hydraulic transmissivity $\theta$ given by:

$$\theta = \rho\,g\,s^3/(12\,\eta) \qquad \text{(Equation 2.2-27)}$$

For example, using the density ($\rho$ = 1000 kg/m³) and the viscosity ($\eta = 10^{-3}$ kg/ms) of water, this equation shows that a spacing s = 1 mm is equivalent to a hydraulic transmissivity of $8 \times 10^{-2}$ m²/s, and a spacing $s = 0.1$ mm is equivalent to a hydraulic transmissivity of $8 \times 10^{-7}$ m²/s. These transmissivity values are consistent with transmissivities of synthetic drainage layers.

The differential equation obtained by Brown et al. is:

$$d(r\, dh/dr)/d\,r = [12\, \eta\, k_C r/(\rho\, g\, s^3)]\,(1 + h/H) \qquad \text{(Equation 2.2-28)}$$

which can be written:

$$(1/r)\,(dh/dr) + d^2h/dr^2 = [12\eta\, k_C/(\rho\, g\, s^3)]\,(1 + h/H) \qquad \text{(Equation 2.2-29)}$$

Combining Equation 2.2-27 and 2.2-29, it appears that Equation 2.2-29 [Brown et al.] is identical to Equation 2.2-23 [Fukuoka, 1986] except for the last term, $h/H$ for Fukuoka and $(1 + h/H)$ for Brown et al. Brown et al. solved this differential function using Bessel functions to interpret results from their laboratory model (see Section 2.2.4.3.2). However, the charts they proposed for field conditions were established with a simplifying assumption: the hydraulic gradient for the vertical flow in soil is one. In other words, they assume that the hydraulic head on top of the low-permeability soil is zero. This assumption is valid only if the hydraulic head on top of the FML is much smaller than the thickness of the low-permeability soil layer. This assumption is:

- always acceptable for bottom liners;
- never acceptable for surface impoundment top liners; and
- probably acceptable in most cases of top liners for landfills.

With the simplifying assumption of a gradient of one in the soil, the differential equation governing the flow becomes, as indicated by Brown et al.:

$$\frac{dh}{dr} = \frac{6\, \eta\, k_C}{\rho\, g\, s^3}\,(r - \pi R^2/r) \qquad \text{(Equation 2.2-30)}$$

which gives the following relationship [Brown et al.]:

$$h + H = [3\ \eta\ k_C\ d^2/(4\ \rho\ g\ s^3)]\ [2\ (2R/d)^2\ \text{Log}\ (2R/d) - (2R/d)^2 + 1] \qquad \text{(Equation 2.2-31)}$$

where: h = hydraulic head on top of the FML; H = thickness of the low-permeability soil layer; $\eta$ = viscosity of the leachate; $k_C$ = hydraulic conductivity of the low-permeability compacted soil; d = diameter of the hole in the FML; $\rho$ = density of the leachate; g = acceleration of gravity; s = spacing between the FML and the low-permeability compacted soil; and R = radius of the wetted area.

Equation 2.2-31 gives the radius of the wetted area if the spacing s between the FML and the low-permeability compacted soil is known. Guidance regarding selection of spacing values can be obtained through backcalculation of Brown et al.'s test results (see Section 2.2.4.3.2).

When the radius R of the wetted area is known, the leakage rate can be determined by using the following equation which derives from Darcy's equation with the assumption that the hydraulic gradient is one in the low-permeability compacted soil:

$$Q = \pi\ R^2\ k_C \qquad \text{(Equation 2.2-32)}$$

The above equations were used by Brown et al. to establish charts giving the leakage rate and the radius of the wetted area (Figures 2.2-9 through 2.2-12). To summarize results presented in these charts and extrapolate or interpolate them, we propose the following equations:

$$Q = 0.7\ a^{0.1}\ k_C{}^{0.88}\ h \qquad \text{(Equation 2.2-33)}$$

$$R = 0.5\ a^{0.05}\ k_C{}^{-0.06}\ h^{0.5} \qquad \text{(Equation 2.2-34)}$$

These empirical equations are only valid with the units indicated: Q = leakage rate ($m^3/s$); a = surface area of FML hole ($m^2$); $k_C$ = hydraulic conductivity of low-permeability compacted soil (m/s); h =

hydraulic head on top of FML (m); and R = radius of wetted area between FML and soil (m).

### 2.2.4.3 Laboratory Models

#### 2.2.4.3.1 Introduction

Tests to evaluate leakage through composite liners due to a hole in the FML were conducted by Fukuoka [1985, 1986] and Brown et al. [no date]. It is important to recognize that neither the Brown et al. tests or Fukuoka tests were developed to model the field condition of leakage through composite liners. The Brown et al. tests were preliminary and conceptual in nature. The Fukuoka tests do not even directly relate to field conditions existing at landfills and surface impoundments. However, both sets of tests (and in particular the Brown et al. tests) can be used to develop an understanding of the mechanics of flow through composite liners and to relate design equations to field conditions.

In both cases, tests were conducted with a FML having a circular hole, and various hole diameters were used in both testing programs. Additional tests by Brown et al. included FML flaws that are not circular such as slits or seam defects. The tests were intended to be full-scale models of the reality since hole size, FML thickness, and (approximately) soil layer thickness were similar to what they are in the field. However, the permeameters used had a limited diameter (e.g., 0.6 m for Brown et al., and 1.5 m for Fukuoka) and the extension of lateral flow between the FML and soil was limited by the walls of the permeameter.

In the tests conducted by Brown et al., the FML was always covered by 0.15 m (6 in.) of gravel to ensure contact between FML and soil, and, in some tests, an additional load up to 160 kPa (3340 psf) (equivalent to 10 m of soil) was applied to evaluate the effect of overburden pressure. In many of the tests conducted by Fukuoka, the FML was not covered, and the only load applied on the FML was the water pressure.

Water heads in Brown et al. tests were up to 1 m, while in Fukuoka tests, they were up to 40 m. Tests by Brown et al. were conducted for landfill applications while Fukuoka was working on the design of a large dam and reservoir.

Fukuoka used only a PVC FML, while Brown et al. considered a variety of FMLs: HDPE, PVC, CSPE, and EPDM, with various thicknesses.

Tests by Fukuoka as well as tests by Brown et al. showed that there is flow between the FML and the soil. Some of the tests conducted by Fukuoka and by Brown et al. included a geotextile between the FML and the soil. With a geotextile, flow between the liners would be expected and the liners do not represent a true composite liner. However, from these tests with geotextiles, some understanding of the effect of an imperfect FML to soil contact can be obtained.

### 2.2.4.3.2 Review of Tests by Brown et al.

These tests are presented in a report by Brown et al. [no date].

- Description of the Tests

Tests were conducted in a 0.6 m (24 in.) diameter permeameter. Hole diameters ranged between 0.8 mm (1/32 in.) and 13 mm (1/2 in.), and non-circular holes such as slits and seam defects were considered.

The FMLs were: HDPE (0.8 mm to 2.5 mm) (30 to 100 mils); PVC (0.5 to 0.8 mm) (20 to 30 mils); CSPE (0.9 to 1.15 mm) (36 to 45 mils); and EPDM (0.8 mm) (30 mils).

In some tests, geotextiles were included between the FML and the soil. The geotextiles were needlepunched nonwovens with a mass per unit area of 250 to 350 $g/m^2$ and a thickness (under no load) on the order of 2.5 to 4 mm.

The soils used were a silty sand ($k = 2 \times 10^{-6}$ m/s), and a clayey silt ($k = 2 \times 10^{-8}$ m/s).

- Approach

The diameter of the permeameter used by Brown et al. was small (0.6 m) and lateral flow could not extend beyond a radius of 0.3 m as it would have in most cases without the limitation imposed by the permeameter walls. Therefore, the calculations presented in Section 2.2.4.2.3 were used to backcalculate the value of the spacing between the FML and soil from the test results. The value of the spacing thus obtained can then be used in similar equations to determine the radius of the wetted area and, therefore, the leakage rate in actual situations where lateral expansion of the flow is not impeded by permeameter walls. The backcalculated spacing values are as follows:

0.02 mm for clayey silt regardless of FML

0.08 mm for silty sand and flexible FML (PVC)

0.15 mm for silty sand and stiff FML (HDPE)

Spacing between the FML and the soil, and, therefore, the leakage rate, appears to increase if the FML stiffness increases (at least in the case of the more permeable soil). It also appears that spacing increases if the soil is coarse, which is illustrated by:

0.02 mm = $d_{10}$ of clayey silt

0.08 mm = $d_{16}$ of silty sand

The above spacing values are related to the case of a FML with 15 cm of gravel overburden. This is an unrealistically low overburden pressure in relation to field conditions. This fact, coupled with the fact that Brown et al.'s tests were short term, implies that the spacing values cited above are somewhat conservative.

Following is a review of the influence of various parameters on test results.

### - Effect of Overburden Pressures

When a compressive stress of 160 kPa (equivalent to 10 m of soil) is applied on a HDPE FML 0.75 mm (30 mil) thick placed on a soil with a hydraulic conductivity of $2 \times 10^{-6}$ m/s, the flow rate through an FML hole is divided by 200 and the backcalculated theoretical spacing between FML and soil is divided by 10 (there are no results for the soil with a hydraulic conductivity of $2 \times 10^{-8}$ m/s).

### - Effect of Flaw Shape

Erratic results were obtained with slits and seam defects on the soil with $k_c = 2 \times 10^{-6}$ m/s:

- Some tests showed that a 50 mm slit or seam defect is often equivalent to a 0.5 to 1 mm diameter circular hole (however other tests showed that a 50 mm seam defect can be equivalent to a 75 mm diameter hole).

- Tests showed that a 150 mm slit or seam is often equivalent to a 75 mm diameter circular hole (which is very different from the 0.5 to 1 mm diameter circular hole indicated above as equivalent to a 50 mm seam defect).

It was difficult to compare slits, seams and circular holes with the $2 \times 10^{-8}$ m/s soil because for that soil there is more lateral flow and permeameter walls disturbed the flow.

### - Conclusions from Brown et al.'s Tests

In order to extrapolate to field conditions, Brown et al. make the following recommendations regarding the values of the spacing between FML and soil to be used in the equations presented in Section 2.2.4.2.3 to evaluate leakage rate and radius of wetted area in actual field conditions where lateral extension of flow is not impeded by wall permeameter:

| soil hydraulic conductivity, $k_c$ (m/s) | FML-soil spacing, s (mm) |
|---|---|
| $10^{-6}$ | 0.15 |
| $10^{-7}$ | 0.08 |
| $10^{-8}$ | 0.04 |
| $10^{-9}$ | 0.02 |

These values are the upper boundary of (or even larger than) the backcalculated spacing values previously given in the discussion of the approach. Also, these spacing values are for the case when there is little or no overburden (e.g., 15 cm of gravel), and they are expected to be larger than in the case when there is a large overburden. Therefore, for these two reasons, leakage rates calculated by Brown et al. are likely to be conservative. It is clear, however, that the results of the Brown et al. study indicate a significant benefit of a composite liner design consisting of a FML upper component and compacted soil lower component.

### 2.2.[illegible].3.3 Review of Tests by Fukuoka

These tests are described in [Fukuoka, 1985; and Fukuoka, 1986]. They were conducted for the design of the lining system for a dam and a reservoir with a maximum water head of 40 m (130 ft). Although these conditions are not representative of hazardous land disposal units, the study conducted by Fukuoka, when combined with the findings of Brown et al., provide a good understanding of the mechanisms governing leakage through composite liners.

- Description of the Test

All tests discussed below were conducted with the following equipment, conditions, and materials: permeameter diameter is 1.5 m (5 ft); water pressure is 200 or 400 kPa (4,000 or 8,000 psf); soil permeability is on the order of $10^{-7}$ to $10^{-6}$ m/s ($10^{-5}$ to $10^{-4}$ cm/s);

soil thickness is 0.45 m (1.5 ft) when no soil cover is placed on the FML and 0.225 m when a 0.225 m (0.75 ft) thick soil cover is placed on the FML; the FML is a 1 mm (40 mil) thick PVC; the geotextile is a needlepunched nonwoven geotextile (mass per unit area 450 g/m² (13 oz/sq. yd), 4 mm (160 mil) thick, permeability 0.001 m/s (0.1 cm/s) under no pressure and 0.0005 m/s (0.05 cm/s) under a 400 kPa (8,000 psf) pressure).

- Tests with FML Alone on Soil (no geotextile, no cover)

In this case, tests show that the diameter of the FML hole needs to be larger than 2 mm (0.08 in.) approximately in order to ensure that free flow through the hole (assuming there is nothing under the FML) is larger than flow rate through soil alone. This indicates that the soil layer has less influence in reducing leakage rate in the case of very small holes than in the case of large holes.

Tests showed that the leakage rate becomes equal to the leakage rate with no FML at all when the diameter of the FML hole is larger than approximately 20 mm (3/4 in.) (Figure 2.2-13). This indicates that leakage flows laterally between the FML and the soil and reaches the walls of the permeameter (diameter 1.5 m (5 ft)) when the diameter of the hole is 20 mm (3/4 in.) or more. This also indicates that the pressure in the liquid located between the FML and soil is the same as the pressure on top of the FML.

Pressure measurements in the soil (Figure 2.2-14a) showed that the full water pressure is applied on top of the soil, which confirms that there is a space between FML and soil where water flows freely. In other words the FML was slightly uplifted by water. (Note that pressure on top of the FML, plus the weight of the FML (specific gravity 1.2) exceeds the pressure under the FML by 2 Pa (0.04 psf). This is an extremely small pressure (i.e., of the order of the pressure exerted by a couple of sheets of paper in dry conditions) and it is easily overcome by the stiffness of the FML, even a FML as flexible as PVC - a PVC FML wrinkle can easily carry a couple of sheets of paper.)

- Tests with FML on Geotextile on Soil

The geotextile had no hole (only the FML had a hole). The geotextile and the FML were not glued together (i.e., the FML was simply laid on the geotextile). (This detail is important in the discussion presented hereafter.)

When FML hole was smaller than 30-50 mm (1-2 in.) approximately, flow rate was approximately 20 times smaller than flow rate through soil alone. In other words, when FML hole diameter was smaller than 30-50 mm, using a geotextile under the FML decreased the flow rate by approximately one order of magnitude or more.

Pressure measurements in the soil in the case of a 20 mm (3/4 in.) diameter FML hole (Figure 2.2-14 b) showed that the water pressure on the soil surface (i.e., under the geotextile) was roughly uniform and 15 times smaller than the uniform pressure in the case without geotextile between FML and soil. This indicates that the head and, consequently, flow rate was 15 times smaller with geotextile than without geotextile, which is consistent with the observations mentioned above.

Pressure measurement in the soil in the case of a 50 mm (2 in.) diameter FML hole (Figure 2.2-15) showed that water pressure on the soil surface was less uniform than in the case of a 20 mm (3/4 in.) diameter FML hole. Pressures were larger in the vicinity of the hole which indicated that there was water flowing in the geotextile within a radius smaller than the radius of the test permeameter.

It may be concluded that FML, geotextile and soil stay in close contact when the FML hole is smaller than 50 mm (2 in.). This appears clearly because:

- if water were accumulating between FML and geotextile, the water pressure on the soil would be uniformly high, almost equal to the water pressure on the FML (i.e., 200 or 400 kPa) (4,000 or 8,000 psf) since geotextile permittivity (i.e., permeability/thickness) is much larger than soil permittivity and, therefore, head loss through geotextile would be small; and

- if water were accumulating between geotextile and soil, both geotextile and FML would [illegible] uplifted and the water pressure on the soil would be equal to the water pressure on the FML (i.e., 200 or 400 kPa (4,000 or 8,000 psf)).

FML, geotextile, and soil stay in close contact because the pressure on top of the FML (200 or 400 kPa) (4,000 or 8,000 psf) is much higher than the pressure below the geotextile. The same would happen with the FML alone (i.e., water pressure on top of the FML would be higher than water pressure under the FML) if the FML were in close contact with the soil. But, if the FML were not in close contact with the soil because of small soil surface irregularities, and if there were preferential channels for the flow of water between the FML and soil, water pressure between FML and soil might become equal to water pressure on top of the FML. If the soil surface were perfectly smooth, and if the FML had no wrinkle, there would be no preferential path for the water: the FML and the soil would stay in close contact (the same way two pieces of polished steel stick to each other because there is no air or water pressure between them).

- Tests with Earth Cover on the FML, but no Geotextile

In this case, the tests (conducted with FML hole diameter of 10 and 20 mm (3/8 and 3/4 in.)) show a flow rate reduction of the order of 40% (i.e., a factor of 1.66) as compared to the case where there is no earth cover on the FML (Figure 2.2-13). The thickness of the earth cover was 0.225 m (0.75 ft), and the thickness of the soil under the

FML was 0.225 m (0.75 ft) (i.e., a total soil thickness of 0.45 m (1.5 ft) as in the tests discussed above).

More tests would be necessary to draw conclusions, such as tests with a permeable cover material and comparable tests with identical low-permeability compacted soil layer thickness under the FML. However, the tests by Fukuoka show that an earth cover, even on a flexible FML such as PVC, does not have a marked effect on leakage rate probably because it is not sufficient to force the FML into soil irregularities.

### 2.2.4.4 Conclusions on Leakage through Composite Top Liners

#### 2.2.4.4.1 Conclusions from Analytical Studies

It appears that the theoretical analyses involved in the apparently simple problem of leakage through a hole in a FML placed on a low permeability soil to form a composite liner are extremely complex.

If perfect contact between the FML and soil is considered, the two-dimensional problem has been solved but the three-dimensional problem still requires research. There is no satisfactory approximate solution and the analytical lower and upper boundaries are too far from the actual solution to give valuable information.

Differential equations have been proposed and some approximate numerical solutions are available for the case of imperfect contact between the FML and soil. To use these equations, it is necessary to know the spacing between the FML and the underlying low-permeability soil. Spacing values backcalculated from model tests are only preliminary and are probably smaller than actual spacing values in the field. The field conditions listed below will affect actual site-specific leakage rates. While the quality of the FML-compacted soil contact was probably better in the laboratory tests than it would be in the field, the laboratory tests to date have been carried out only for short durations and at unrealistically low levels of overburden pressure. Field conditions affecting spacing between the FML component and the soil component of a composite liner are the following:

- subgrade surface preparation;
- FML wrinkles;
- overburden; and
- time.

As a result, actual leakage rates in the field are likely to differ from those calculated using equations incorporating FML-soil spacings backcalculated from model tests. Also, it is likely that there will be some spatial variation throughout the liner.

#### 2.2.4.4.2 Conclusion from Model Tests

Tests show that, in all cases where a FML is placed in direct contact with a low permeability soil, some liquid that has passed through a hole in the FML flows laterally in the space between the FML and the underlying soil. Tests show that, as a result of lateral flow, leakage rates observed are higher than leakage rates which would be obtained if there was a perfect contact between the FML and the underlying soil. The degree of contact between the FML and soil in the model tests can be considered good (smooth soil surface, no cracks in clay) but not perfect since flow takes place between the FML and the soil.

From a construction standpoint, it is recommended to make every effort to ensure a good contact between FML and low-permeability soil which includes: (i) having a low-permeability soil with a smooth surface and no cracks; and (ii) wrinkles in the FML. Ideally, the FML should be sprayed on the low permeability soil instead of being made in a plant and transported to the site: in this case, the contact may be very good.

From a design standpoint, it is necessary to take into account the flow of leachate between the FML and the soil for leakage evaluation as well as for any other appropriate design consideration such as damage caused to the soil layers by liquid flowing in the space between the FML and the underlying soil layer.

Although the tests provided a good understanding of the mechanisms involved, the diameter of the permeameter and test conditions used by Brown et al. and Fukuoka limit the usefulness of the test results for the development of design recommendations. Although extrapolation of test data to field conditions was done by Brown et al. using a sound theoretical analysis, test conditions were too far from actual conditions to ensure that extrapolated values are adequate.

In spite of their limitations, the preliminary tests described in this report show that composite liners are significantly more effective than low-permeability compacted soil alone or FML alone. However, the test results indicate that a FML in perfect contact with a low-permeability soil (i.e., a FML sprayed on the soil) would exhibit even better performance than composite liners made by unrolling FMLs directly onto the soil.

### 2.2.4.4.3 Conclusions for Leakage Rate Evaluation

- Review of Methods for Leakage Rate Evaluation

A series of methods have been discussed to evaluate leakage rate through a composite liner due to a hole in the FML. These methods can be ranked as follows:

- An absolute minimum of the leakage rate is given by the vertical flow equation assuming perfect contact between the FML and the underlying soil (Equation 2.2-19).

- An approximate value (possibly an underestimate) of the leakage rate in case of perfect contact between the FML and the underlying soil is given by Equation 2.2-21. Since this equation has not been tested, it is appropriate to have the absolute minimum mentioned above to make sure that no absurd result is considered.

- Leakage rate obtained using charts prepared by Brown et al. on the basis of their tests (Figures 2.2-9 through 2.2-12) or the empirical equations we have proposed to summarize these charts (Equations 2.2-33 and 2.2-34) may be smaller than actual leakage rate because in the field FMLs have at least some wrinkles and subgrade preparation may not be as good as in the model tests, thereby allowing more flow between the FML and the soil in the field than in the models. However, a counteracting influence is that the overburden pressure in the model tests was well below overburden pressures representative of field conditions.

- Finally, leakage through a hole in a FML alone (i.e., with nothing underneath it) is certainly much larger than leakage through a composite liner with the same FML hole, even in field conditions with a far from perfect contact between the FML and the underlying soil. This case, therefore, provides an absolute maximum of the leakage rate.

Leakage rate through a hole in a FML alone is given by Bernouilli's equation (Equation 2.2-13). By combining Equations 2.2-12 and 2.2-32, it appears that, if the spacing between the FML and the soil is large enough to ensure free flow, the radius of the wetted area is given by:

$$\pi R^2 k_C = 0.6 a \sqrt{2 g h} \qquad \text{(Equation 2.2-35)}$$

hence:

$$R = 0.44\ a^{0.5}\ (2\ g\ h)^{0.25}\ k_c^{-0.5} \quad \text{(Equation 2.2-36)}$$

and, in the case of a circular hole:

$$R = 0.39\ d\ (2\ g\ h)^{0.25}\ k_c^{-0.5} \quad \text{(Equation 2.2-37)}$$

where: $R$ = radius of the wetted area; $a$ = hole area; $d$ = hole diameter; $g$ = acceleration of gravity; $h$ = hydraulic head on top of FML; and $k_c$ = hydraulic conductivity of the low-permeability compacted soil underlying the FML. Recommended SI units are: $R$ (m), $a$ ($m^2$), $d$ (m), $g$ ($m/s^2$), $h$ (m), and $k_c$ (m/s).

A summary of pertinent equations is presented in Table 2.2-11.

- Leakage Rate and Radius Graphs

Because of the uncertainties in the analyses as well as the wide variety of contact conditions, it is appropriate in each given case to plot leakage rates obtained with all the methods described above in order to make interpolations. It is also appropriate to use a semi-logarithmic scale for the plot since leakage rates vary within a range of several orders of magnitude, as is usually the case in hydraulic problems. The graph in Figure 2.2-16 has been established with a 1 $cm^2$ hole, which, as described previously, represents a large hole (larger than current BDAT capabilities), possibly representative of a design flaw or operational damage. This standard hole is used herein for design calculations, as indicated in Section 2.2.3.3.6. The graph in Figure 2.2-16 has been established for a hydraulic head of 30 mm (0.1 ft) on top of the FML. Numerical values used to establish the graph in Figure 2.2-16 are given in Table 2.2-12.

Similarly, a graph can be established for the radii of wetted areas (i.e., the area covered by leakage flowing between the FML and the low-permeability compacted soil, before it flows into the compacted soil) obtained with all the methods described above and summarized in Table 2.2-11. The radius graph related to a hydraulic head of 30 mm (0.1 ft) on top of the FML is given in Figure 2.2-17.

Similar graphs were established for a head of 0.3 m (1 ft) and 3 m (10 ft). These graphs are presented in Figures 2.2-18 through 2.2-21. Numerical values used to establish Figures 2.2-18 through 2.2-21 are given in Tables 2.2-13 and 2.2-14.

- Use of Leakage Rate and Radius Graph

The leakage rate graph permits the determination of the leakage rate for any given field condition by interpolation between the best case and the worst case:

- In the best case: (i) the soil is well compacted, flat and smooth, has not been deformed by rutting due to construction equipment, and has no clods nor cracks; and (ii) the FML is flexible and has no wrinkles.

- In the worst case: (i) the soil is poorly compacted, has an irregular surface, and is cracked; and (ii) the FML is stiff and exhibits a pattern of large, connected wrinkles. (Note: this worst case condition is improbable with proper construction quality assurance.)

The conditions in the best case may be as good as the conditions in the tests by Brown et al. discussed in Section 2.2.4.3. Therefore, on the graphs, the best case for field conditions is represented by the vertical line corresponding to test results.

In order to locate the worst field case (which is unrealistic with current construction quality assurance standards) for a range of hydraulic heads on the top liner of 0.03 m (0.1 ft) to 3 m (10 ft), we have assumed that the radius of the wetted area cannot exceed a value ranging from 3 to 30 m (10 to 100 ft). A value of the compacted soil hydraulic conductivity of $10^{-8}$ m/s ($10^{-6}$ cm/s) was also assumed. This value of $k_d$ has been selected to represent a worst case condition. The location of the worst case line thus obtained shows that the conditions in the worst case are still much better than the case of free flow through holes in the FML. Free flow is an extreme case which is possible only if the FML is very far from the low-permeability compacted soil over a very large area (radius of 10 to 100 m), which is practically impossible.

Between the best field case and the worst field case we have selected a vertical line representing good field conditions and a vertical line representing poor field conditions. As a result, it appears in Figure 2.2-16 that, for a head of 30 mm (0.1 ft), a leakage rate of 0.08 liters/day (0.02 gallon/day) corresponds to good field conditions and a hydraulic conductivity of $k_c = 10^{-9}$ m/s ($10^{-7}$ cm/s) for the low-permeability compacted soil underlying the FML. Poor field conditions were based on poor FML contact and a hydraulic conductivity, $k_d = 10^{-8}$ m/s ($10^{-6}$ cm/s). Under these conditions (which might represent the worst possible realistic field conditions) the leakage rate value obtained from Figure 2.2-16 is 4 liters/day (1 gallon/day).

Figure 2.2-21 gives the radius of the wetted area in the case of a head of 3 m (10 ft), which is a reasonable maximum of the hydraulic head at hazardous waste surface impoundments. (The wetted area is the area where leakage flows between the FML and the underlying compacted soil before it seeps into the compacted soil.) Figure 2.2-21 shows that the radius of the wetted area is between 3 and 24 m (10 and 80 ft) depending on the hydraulic conductivity of the compacted soil and the quality of the contact between the FML and the compacted soil. In other words, the large leakage estimated in the case of surface impoundments corresponds to a large wetted area. This indicates that leakage rate through top liners in the case of surface impoundments can be decreased if flow between the FML and the underlying compacted soil is restricted. Therefore, an overburden on the FML will be beneficial since it will impede lateral flow between FML and underlying soil, thereby decreasing leakage. It is preferable that the overburden be provided by a low-permeability soil which will further decrease leakage rate.

- Leakage Rate due to Permeation and Holes

Leakage rates through composite liners due to hole in the FML, obtained from Figure 2.2-16 (head of 30 mm (0.1 ft)), Figure 2.2-18 (head of 0.3 m (1 ft)), and Figure 2.2-19 (head of 3 m (10 ft)), are summarized in Table 2.2-15, which also gives leakage rates due to permeation obtained from Table 2.2-10. (Although Table 2.2-10 was established for a FML alone, leakage rates due permeation from Table 2.2-10 can be used for composite liners since leakage rate due to permeation should not be significantly affected by the underlying soil because all soil are very pervious as compared to FMLs.)

To the best of our knowledge, Table 2.2-15 summarizes the best demonstrated available technology on leakage rate through composite top liners. This table shows that leakage rates through top composite liners can be larger in the case of surface impoundments than in the case of landfills or waste piles.

### 2.2.5 Conclusions on Leakage through Top Liners

General conclusions are drawn hereafter from all discussions presented in Section 2.2, which is a thorough review of theoretical analyses, laboratory tests, and field data. However, it should be pointed out that these are only tentative conclusions since they should be subjected to peer review. The tentative conclusions are as follows.

#### 2.2.5.1 Defects and Quality Assurance

With good quality assurance, it is reasonable to expect 3 to 5 FML defects per hectare (one or two seam defects per acre). These defects can be caused by inadequate seaming (which is the most common type of defect) and FML defects caused by puncture, tear, excessive stresses, etc. Defects may also be due to inadequate FML connections to sumps and other appurtenances which are often problem areas. Also, the FML may undergo excessive stresses in the vicinity of these connections that may cause defects to develop. Under the very best conditions with excellent design, installation and construction quality assurance, a frequency of FML defects below 3 to 5 per hectare (1 to 2 per acre) may be possible.

The leakage rate values summarized in Table 2.2-16 are based on a FML defect frequency of 3 per hectare (1 per acre) and on a "standard" hole size of 1 $cm^2$ (0.16 $in^2$.). It must be remembered that the results presented in Table 2.2-16 are for a properly designed and constructed land disposal unit in which: (i) good quality assurance is provided; (ii) extreme care is taken for FML connections to sumps and other appurtenances; and (iii) a proper design minimizes the risk of excessive stresses, which could generate holes much larger than 1 $cm^2$ (0.16 $in^2$.).

### 2.2.5.2 Summary of Leakage Rate Values

Table 2.2-16 summarizes leakage rates through top liners. This table has been obtained by combining Table 2.2-10 for FMLs alone with Table 2.2-15 for composite liners. To the best of our knowledge, this table represents a reasonable summary of the rates of leakage through properly designed and constructed top liners.

This table has been established with the following assumptions:

- The FML is 1 mm (40 mil) thick and has one standard defect with a surface area of 1 $cm^2$ (0.16 $in^2$.) per 4,000 $m^2$ (acre).
- The low-permeability compacted soil underlying the FML has a thickness of 0.9 m (1 ft) and a hydraulic conductivity of $10^{-9}$ m/s ($10^{-7}$ cm/s) in the case of good FML/compacted soil contact and $10^{-8}$ ($10^{-6}$ cm/s) in the case of poor FML/compacted soil contact.

The three hydraulic heads used in Table 2.2-16 represent the following conditions:

- 0.03 m (0.1 ft) is assumed to be the average head acting on the top liner of a landfill or waste pile with a well designed and constructed leachate collection and removal system (LCRS), above the top liner;

- 0.3 m (1 ft) is the maximum head considered in the design of the leachate collection and removal system (LCRS) of a landfill or waste pile; and

- 3 m (10 ft) is assumed to be the maximum head on the top liner of a surface impoundment.

### 2.2.5.3 Comments on Leakage Rate Values

It appears from Table 2.2-16 that leakage rates through FML top liners are relatively high if there is one hole per 4,000 $m^2$ (acre) in the FML: typically 30 to 300 Ltd (gpad) in landfills and waste piles (and up to 1,000 Ltd (gpad) in the extreme) and 300 to 3,000 Ltd (gpad) in surface impoundments. Since it is impossible to guarantee that there will be no hole in a FML, owners and operators may increasingly consider the use of a composite top liner.

Table 2.2-16 shows that leakage rates in the case of a landfill with a composite top liner can be small (i.e., almost always less than 10, and usually less than 1 Ltd (gpad)). However, it should be kept in mind that such low leakage rates can be achieved only if the lining system is constructed with very good quality assurance and if the FML is not subjected to excessive stresses likely to cause a large breach.

Table 2.2-16 also shows that, even with a composite top liner the leakage rate may remain high (e.g., from 10 to 50 Ltd (gpad)) in a surface impoundment with a hydraulic head of 3 m (10 ft). As indicated in Section 2.2.4.4.3, leakage rates can be decreased if the FML is covered with a layer of soil, preferably a soil with a low permeability. It also appears that leakage rates due to permeation through the FML may not be negligible in the case of surface impoundments.

Table 2.2-1. Summary of Typical FMLs

| CATEGORY | POLYMER | SYMBOL[a] |
|---|---|---|
| Thermoplastics | Polyvinyl Chloride | PVC |
| | Oil-resistant PVC | PVC-OR |
| | Thermoplastic Nitrile-PVC | TN-PVC |
| | Ethylene Interpolymer Alloy | EIA |
| Crystalline Thermoplastics | Low Density Polyethylene | LDPE |
| | Linear Low Density Polyethylene | LLDPE |
| | High Density Polyethylene | HDPE |
| | High Density Polyethylene-Alloy | HDPE-A |
| | Polypropylene | PP |
| | Elasticized Polyolefin | ELPO |
| Thermoplastic Elastomers | Chlorinated Polyethylene | CPE |
| | Chlorinated Polyethylene-Alloy | CPE-A |
| | Chlorosulfonated Polyethylene ("Hypalon") | CSPE |
| | Thermoplastic Ethylene-Propylene Diene Monomer | T-EPDM |
| Elastomers | Isoprene-Isobutylene Rubber ("Butyl Rubber") | IIR |
| | Ethylene-Propylene Diene Monomer | EPDM |
| | Polycholoroprene ("Neoprene") | CR |
| | Epichlorohydrin Rubber | CO |

[a] symbols consistent with those used by the National Sanitation Foundation (NSF) Joint Committee of Flexible Membrane Liners (FML)

Note: Polymers are usually compounded with various additives such as fillers, fibers, carbon black, plasticizers, stabilizers, antioxidants, fungicides, and other polymers. These additives perform various functions without altering the very low permeability of the base product.

Table 2.2-2. Typical values of soil hydraulic conductivities [Giroud, 1975].

| m/s | 1 | $10^{-2}$ | $10^{-4}$ | $10^{-6}$ | $10^{-8}$ | $10^{-10}$ | $10^{-12}$ |
|---|---|---|---|---|---|---|---|
| cm/s | $10^{2}$ | 1 | $10^{-2}$ | $10^{-4}$ | $10^{-6}$ | $10^{-8}$ | $10^{-10}$ |
| feet/year | $10^{8}$ | $10^{6}$ | $10^{4}$ | $10^{2}$ | 1 | $10^{-2}$ | $10^{-4}$ |
| Soil with a small coefficient of uniformity that does not contain particles finer than indicated | Gravel | Sand (coarse, medium) | | Fine Sand<br>Silt<br>Cracked Clay | | Non Cracked Clay | |
| Soil mixtures that do not contain particles finer than indicated | | Gravel + Sand | | Sand + Silt | Fine Sand (or silt) + Clay | | |

Table 2.2-3. Values of the migration coefficient, μ, obtained from permeability tests conducted at the University of Grenoble (France) with the apparatus shown in Figure 2.2-1.

| | hydraulic head, h, in m | | | | | |
|---|---|---|---|---|---|---|
| FML Type | 5 | 10 | 25 | 50 | 75 | 100 |
| CSPE | | $3.8\text{x}10^{-12}$ | | $5.0\text{x}10^{-12}$ | | $5.5\text{x}10^{-12}$ |
| Butyl | | $7.7\text{x}10^{-12}$ | | $3.9\text{x}10^{-12}$ | | $3.1\text{x}10^{-12}$ |
| Butyl | $3.5\text{x}10^{-16}$ | $1.7\text{x}10^{-13}$ | $1.9\text{x}10^{-12}$ | $2.9\text{x}10^{-13}$ | | $3.0\text{x}10^{-13}$ |
| EPDM | | $1.1\text{x}10^{-12}$ | | $2.3\text{x}10^{-12}$ | | $2.2\text{x}10^{-12}$ |
| PVC | | $1.7\text{x}10^{-12}$ | | $2.5\text{x}10^{-12}$ | | $1.1\text{x}10^{-12}$ |
| PVC | | $1.6\text{x}10^{-12}$ | | $2.1\text{x}10^{-12}$ | | $4.4\text{x}10^{-13}$ |
| PVC | | $8.1\text{x}10^{-13}$ | | $2.0\text{x}10^{-12}$ | | $1.0\text{x}10^{-12}$ |
| Asphaltic | $4.2\text{x}10^{-13}$ | $7.4\text{x}10^{-13}$ | $6.7\text{x}10^{-13}$ | $6.5\text{x}10^{-13}$ | $7.4\text{x}10^{-13}$ | |
| Asphaltic | | $1.6\text{x}10^{-13}$ | $3.2\text{x}10^{-13}$ | $6.5\text{x}10^{-13}$ | $4.5\text{x}10^{-13}$ | |
| | Values of μ in $m^2/s$ | | | | | |

Table 2.2-4. Water vapor transmission (WVT) rates of FMLs from [Haxo et. al., 1984] and values of the coefficient of migration derived from WVT values using Equation 2.2-9 (See also Table 2.2-5). All these tests were conducted at 23°C with a relative humidity difference of 50%, which is equivalent to a pressure of 1.4 kPa, i.e., a head of 0.14 m of water.

| Polymer | Thickness, (mm) | Water Vapor Transmission WVT, ($g/m^2$.day) | Coefficient of migration $\mu$ ($m^2/s$) |
|---|---|---|---|
| Butyl rubber | 0.85 | 0.384 | $3.8 \times 10^{-15}$ |
| | 0.85 | 0.020 | $2.0 \times 10^{-16}$ |
| | 1.85 | 0.097 | $2.1 \times 10^{-15}$ |
| CPE | 0.53 | 0.643 | $3.9 \times 10^{-15}$ |
| | 0.79 | 1.400 | $1.2 \times 10^{-14}$ |
| | 0.79 | 0.320 | $2.9 \times 10^{-15}$ |
| | 0.85 | 0.264 | $2.6 \times 10^{-15}$ |
| | 0.94 | 0.305 | $2.2 \times 10^{-15}$ |
| | 0.97 | 0.643 | $7.2 \times 10^{-15}$ |
| CSPE | 0.74 | 0.333 | $2.9 \times 10^{-15}$ |
| | 0.76 | 0.663 | $5.8 \times 10^{-15}$ |
| | 0.89 | 0.438 | $4.5 \times 10^{-15}$ |
| | 0.91 | 0.748 | $7.9 \times 10^{-15}$ |
| | 0.94 | 0.422 | $4.6 \times 10^{-15}$ |
| | 1.07 | 0.252 | $3.1 \times 10^{-15}$ |
| ELPO | 0.72 | 0.142 | $1.2 \times 10^{-15}$ |
| CO | 1.160 | 20.18 | $2.7 \times 10^{-13}$ |
| | 1.650 | 14.30 | $2.7 \times 10^{-13}$ |
| EPDM | 0.51 | 0.270 | $1.6 \times 10^{-15}$ |
| | 0.94 | 0.190 | $2.1 \times 10^{-15}$ |
| | 1.70 | 0.172 | $3.4 \times 10^{-15}$ |

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Table 2.2-4, continued

| | | | |
|---|---|---|---|
| Neoprene | 0.51 | 0.304 | $1.8 \times 10^{-15}$ |
| | 0.91 | 0.473 | $5.0 \times 10^{-15}$ |
| | 1.27 | 0.429 | $6.3 \times 10^{-15}$ |
| | 1.59 | 0.237 | $4.4 \times 10^{-15}$ |
| Nitrile rubber | 0.76 | 5.51 | $4.8 \times 10^{-14}$ |
| PB[a] | 0.69 | 0.084 | $6.7 \times 10^{-16}$ |
| PEEL[b] | 0.20 | 10.50 | $2.4 \times 10^{-14}$ |
| LDPE | 0.76 | 0.0573 | $5.0 \times 10^{-16}$ |
| HDPE | 0.80 | 0.0172 | $1.6 \times 10^{-16}$ |
| | 2.44 | 0.0062 | $1.8 \times 10^{-16}$ |
| HDPE-A | 0.86 | 0.0472 | $4.7 \times 10^{-16}$ |
| PVC | 0.28 | 4.42 | $1.4 \times 10^{-14}$ |
| | 0.51 | 2.97 | $1.7 \times 10^{-14}$ |
| | 0.76 | 1.94 | $1.7 \times 10^{-14}$ |
| | 0.79 | 1.85 | $1.7 \times 10^{-14}$ |
| PVC-E | 0.91 | 2.78 | $2.9 \times 10^{-14}$ |
| PVC-OR | 0.83 | 4.17 | $4.0 \times 10^{-14}$ |
| Saran Film | 0.013 | 0.563 | $8.5 \times 10^{-17}$ |

[a] PB = polybutylene.
[b] PEEL = polyester elastomer.
(Other symbols are defined in Table 2.2-1)

Table 2.2-5. Water vapor transmission (WVT) rates of FMLs [Rogers, ] and values of the coefficient of migration derived from WVT values using Equation 2.2-9. (See also Table 2.2-4.)

| FML Type | Reference Pressure $p$ (kPa) | Water Vapor Transmission WVT (g/m$^2$.day) | Reference Thickness $T$ (mm) | Coefficient of Migration $\mu$ (m$^2$/s) |
|---|---|---|---|---|
| Hypalon | 6.4 | 161 | 0.025 | $4.6\times10^{-14}$ |
| Butyl | 6.4 | 26 | 0.025 | $7.5\times10^{-15}$ |
| PVC | 6.1 | 32 | 0.025 | $9.2\times10^{-15}$ |
| HDPE 0.92* | 6.4 | 28 | 0.025 | $8.1\times10^{-15}$ |
| 0.94* | 5.8 | 14 | 0.025 | $4.1\times10^{-15}$ |
| 0.95* | 6.1 | 6.7 | 0.025 | $1.9\times10^{-15}$ |
| 0.96* | 5.8 | 4 | 0.025 | $1.1\times10^{-15}$ |

Notes: (i) the test pressure, $p$, is derived from the test relative humidity difference using Equation 2.2-6; (ii) a 6 kPa pressure is equivalent to a water head of 0.6 m (2 ft); (iii) the values with (*) refer to the polymer density in g/cm$^3$.

Table 2.2-6. Summary of values of the coefficient of migration, μ, from Tables 2.2-3, 2.2-4 and 2.2-5.

| Hydraulic head | FML Type | | |
|---|---|---|---|
| h | CSPE | PVC | HDPE |
| 0.14 m | $5\text{x}10^{-15}$ | $1.7\text{x}10^{-14}$ | $1.7\text{x}10^{-16}$ |
| 0.6 m | $4.6\text{x}10^{-14}$ | $9.2\text{x}10^{-16}$ | $4.1\text{x}10^{-15}$ |
| 10 m | $3.8\text{x}10^{-12}$ | $1.6\text{x}10^{-12}$ | - |
| 50 m | $5.0\text{x}10^{-12}$ | $2.0\text{x}10^{-12}$ | - |
| 100 m | $5.5\text{x}10^{-12}$ | $1.0\text{x}10^{-12}$ | - |
| | Values of coefficient of migration, μ ($m^2/s$) | | |

Table 2.2-7. Values of coefficient of migration resulting from extrapolations and interpolations in Figure 2.2-3.

| | Hydraulic head in m (ft) | | | | |
|---|---|---|---|---|---|
| FML Type | 0 m (0 ft) | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) | > 10 m (> 30 ft) |
| CSPE | 0 | $3.5 \times 10^{-16}$ | $1.5 \times 10^{-14}$ | $6 \times 10^{-13}$ | $6 \times 10^{-12}$ |
| HDPE | 0 | $1.5 \times 10^{-17}$ | $1 \times 10^{-15}$ | $7 \times 10^{-14}$ | $1 \times 10^{-12}$ |
| | Values of coefficient of migration, $\mu$, in $m^2/s$ | | | | $\mu_{max}$ |

Table 2.2-8. Values of rate of leakage due to permeation through FML derived from values of coefficient of migration given in Table 2.2-7, using Equation 2.2-4 and assuming an FML thickness of 1 mm (40 mils).

| | Hydraulic head in m (ft) | | | | |
|---|---|---|---|---|---|
| FML Type | 0 m (0 ft) | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) | > 10 m (> 30 ft) |
| CSPE | 0 | 0.035 | 1.5 | 60 | 600 |
| HDPE | 0 | 0.0015 | 0.1 | 7 | 100 |
| | Values of leakage rate in liters/1000m²/day (Ltd) or gallons/acre/day (gpad) | | | | |

Table 2.2-9. Leakage rate due to pinholes and holes in an FML placed on a very pervious medium such as a drainage layer. Note: in the case of pinholes, FML thickness is 1 mm (40 mils) while in the case of holes, leakage rate is independent of FML thickness.

| | | Hydraulic head | | |
|---|---|---|---|---|
| | Defect diameter | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| Pinholes | 0.1 mm (0.004 in.) | 0.06 (0.015) | 0.6 (0.15) | 6 (1.5) |
| | 0.3 mm (0.012 in.) | 5 (1) | 50 (13) | 500 (130) |
| Holes (*) | 2 mm (0.08 in.) | 125 (30) | 400 (100) | 1250 (300) |
| | 11.3 mm (0.445 in.) | 1,260 (330) | 4,000 (1,000) | 12,600 (3,300) |
| | | Values of leakage rate in liters/day (gallons/day) | | |

(*) The 11.3 mm diameter circular hole has a surface area of 1 $cm^2$.

Table 2.2-10. Leakage rate through a FML top liner assuming one pinhole, small hole, or standard hole per 4,000 $m^2$ (acre). This table has been established by combining Tables 2.2-8 and 2.2-9 and rounding up the numbers. The considered pinhole has a diameter of 0.1 mm (0.004 in.), the small hole has a diameter of 2 mm (0.08 in.), and the standard hole has a surface area of 1 $cm^2$ (0.16 $in.^2$), i.e., a diameter of 11.3 mm (0.445 in.).

| | Hydraulic head | | |
|---|---|---|---|
| | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| Permeation | 0.001 | 0.1 | 10 |
| Pinhole | 0.01 | 0.1 | 1 |
| Small hole | 30 | 100 | 300 |
| Standard Hole | 300 | 1,000 | 3,000 |

Values of leakage rate in liters/1000$m^2$/day (Ltd) or gallons/acre/day (gpad)

Table 2.2-11. Summary of equations giving leakage rate, Q, and radius of wetted area, R, for composite liners when there is a hole in the FML.

---

ABSOLUTE MINIMUM (MIN) in Figures 2.2-16 through 2.2-21
(Vertical flow)

$Q = k_c\ a\ (h + H)/H$ (Equation 2.2-19)

$R = d/2$

---

PERFECT CONTACT (P.C.) in Figures 2.2-16 through 2.2-21
(Approximate value of Q given by radial flow)

$Q = \pi\ k_c\ h\ d$ (Equation 2.2-21)

$R =$ unknown

---

EXCELLENT CONTACT (TEST) in Figures 2.2-16 through 2.2-21
(Empirical equations from model tests)

$Q = 0.7\ a^{0.1}\ k_c^{0.88}\ h$ (Equation 2.2-33)

$R = 0.5\ a^{0.05}\ k_c^{-0.06}\ h^{0.5}$ (Equation 2.2-34)

---

LARGE SPACE BETWEEN FML AND SOIL (MAX) in Figures 2.2-16 through 2.2-21
(Q given by Bernouilli's equation)

$Q = C\ a \sqrt{2gh} = 0.6\ a \sqrt{2gh}$ (Equation 2.2-13)

$R = 0.39\ d\ (2\ g\ h)^{0.25}\ k_c^{-0.5}$ (Equation 2.2-37)

---

where: $k_c$ = hydraulic conductivity of low-permeability compacted soil underlying the FML; a = area of hole in FML; h = hydraulic head on FML; H = thickness of compacted soil layer; d = diameter of hole in FML; and g = acceleration of gravity. Recommended SI units: $k_c$ (m/s), a ($m^2$), h, H, and (m); and g ($m/s^2$). These units are mandatory for the two empirical equations.

Table 2.2-12. Numerical values used to establish the graphs presented in Figures 2.2-16 and 2.2-17. This table has been established for a hydraulic head of 30 mm (0.1 ft) on top of the FML, a hole area of 1 $cm^2$ (0.16 $in^2$.), and a low-permeability compacted soil thickness of 0.9 m (3 ft).

| | | Hydraulic Conductivity of Compacted Soil Underlying the FML | | | |
|---|---|---|---|---|---|
| | Case | Equation | $10^{-7}$ m/s | $10^{-8}$ m/s | $10^{-9}$ m/s |
| Leakage Rate g ($m^3/s$) | Absolute minimum | 2.2-19 | $1.0x10^{-11}$ | $1.0x10^{-12}$ | $1.0x10^{-13}$ |
| | Perfect contact (approximate theory) | 2.2-21 | $1.1x10^{-10}$ | $1.1x10^{-11}$ | $1.1x10^{-12}$ |
| | Good contact (model tests) | 2.2-33 | $5.8x10^{-9}$ | $7.6x10^{-10}$ | $1.0x10^{-10}$ |
| | Free flow (Bernouilli's equation) | 2.2-13 | $4.6x10^{-5}$ | $4.6x10^{-5}$ | $4.6x10^{-5}$ |
| Radius of Wetted Area R (m) | Absolute minimum (hole radius) | R = d/2 | 0.0056 | 0.0056 | 0.0056 |
| | Perfect contact (unknown) | | ≈0.032(*) | ≈0.032(*) | ≈0.032(*) |
| | Good contact (model tests) | 2.2-34 | 0.14 | 0.17 | 0.19 |
| | Free flow | 2.2-37 | 12 | 38 | 122 |

(*) Value obtained by interpolation in Figure 2.2-17.

Table 2.2-13. Numerical values used to establish the graphs presented in Figures 2.2-8 and 2.2-19. This table has been established for a hydraulic head of 0.3 m (1 ft) on top of the FML, a hole area of 1 $cm^2$ (0.16 $in^2$.), and a low-permeability compacted soil thickness of 0.9 m (3 ft).

| | | Hydraulic Conductivity of Compacted Soil Underlying the FML | | | |
|---|---|---|---|---|---|
| | Case | Equation | $10^{-7}$ m/s | $10^{-8}$ m/s | $10^{-9}$ m/s |
| Leakage Rate g ($m^3/s$) | Absolute minimum | 2.2-19 | $1.3 \times 10^{-11}$ | $1.3 \times 10^{-12}$ | $1.3 \times 10^{-13}$ |
| | Perfect contact (approximate theory) | 2.2-21 | $1.1 \times 10^{-9}$ | $1.1 \times 10^{-10}$ | $1.1 \times 10^{-11}$ |
| | Good contact (model tests) | 2.2-33 | $5.8 \times 10^{-8}$ | $7.6 \times 10^{-9}$ | $1.0 \times 10^{-9}$ |
| | Free flow (Bernouilli's equation) | 2.2-13 | $1.5 \times 10^{-4}$ | $1.5 \times 10^{-4}$ | $1.5 \times 10^{-4}$ |
| Radius of Wetted Area R (m) | Absolute minimum (hole radius) | R = d/2 | 0.0056 | 0.0056 | 0.0056 |
| | Perfect contact (unknown) | | ~0.045(*) | ~0.045(*) | ~0.045(*) |
| | Good contact (model tests) | 2.2-34 | 0.45 | 0.52 | 0.60 |
| | Free flow | 2.2-37 | 22 | 69 | 217 |

(*) Value obtained by interpolation in Figure 2.2-19.

Table 2.2-14. Numerical values used to establish the graphs presented in Figures 2.2-20 and 2.2-21. This table has been established for a hydraulic head of 3 m (10 ft) on top of the FML, a hole area of 1 $cm^2$ (0.16 $in^2$.), and a low-permeability compacted soil thickness of 0.9 m (3 ft).

| | | | Hydraulic Conductivity of Compacted Soil Underlying the FML | | |
|---|---|---|---|---|---|
| | Case | Equation | $10^{-7}$ m/s | $10^{-8}$ m/s | $10^{-9}$ m/s |
| Leakage Rate g ($m^3/s$) | Absolute minimum | 2.2-19 | $4.3 \times 10^{-11}$ | $4.3 \times 10^{-12}$ | $4.3 \times 10^{-13}$ |
| | Perfect contact (approximate theory) | 2.2-21 | $1.1 \times 10^{-8}$ | $1.1 \times 10^{-9}$ | $1.1 \times 10^{-10}$ |
| | Good contact (model tests) | 2.2-33 | $5.8 \times 10^{-7}$ | $7.6 \times 10^{-8}$ | $1.0 \times 10^{-8}$ |
| | Free flow (Bernouilli's equation) | 2.2-13 | $4.6 \times 10^{-4}$ | $4.6 \times 10^{-4}$ | $4.6 \times 10^{-4}$ |
| Radius of Wetted Area R (m) | Absolute minimum (hole radius) | R = d/2 | 0.0056 | 0.0056 | 0.0056 |
| | Perfect contact (unknown) | | -0.11(*) | -0.11(*) | -0.11(*) |
| | Good contact (model tests) | 2.2-34 | 1.4 | 1.7 | 1.9 |
| | Free flow | 2.2-37 | 39 | 122 | 386 |

(*) Value obtained by interpolation in Figure 2.2-21.

Table 2.2-15. Leakage rates through composite liners. Leakage due to permeation is obtained from Table 2.2-10 and leakage due to holes is obtained from Figures 2.2-16, 2.2-17 and 2.2-18, as a function of the quality of contact between the FML component and the compacted soil component of the top liner. This table has been established with: hole frequency = 1 per 4000$m^2$ (1 per acre); hole area = 1 $cm^2$ (0.16 $in^2$.); compacted soil thickness = 0.9 m (3 ft); compacted soil hydraulic conductivity $10^{-8}$ m/s ($10^{-6}$ cm/s); and FML thickness = 1 mm (40 mils).

| Quality of contact | Leakage mechanism | Hydraulic head, h | | |
|---|---|---|---|---|
| | | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| Good | Permeation | 0.001 | 0.1 | 10 |
| | Hole | 0.02 | 0.2 | 3 |
| | TOTAL | 0.02 | 0.3 | 13 |
| Poor | Permeation | 0.001 | 0.1 | 10 |
| | Hole | 1 | 8 | 50 |
| | TOTAL | 1 | 8 | 60 |

Values of leakage rate in Ltd or gpad

Table 2.2-16. Leakage rates through top liners. This table has been obtained by combining Tables 2.2-10 and 2.2-15. The small hole has a diameter of 2 mm (0.08 in.). The standard hole has a surface area of 1 cm² (0.16 in²). The frequency of holes is 1 per 4000m² (1 per acre). The thickness of the compacted soil layer is 0.9 m (3 ft) and its hydraulic conductivity is $10^{-8}$ m/s ($10^{-6}$ cm/s). Note: Ltd = liter/1000m²/day; gpad = gallons/acre/day; 1 Ltd = 1.1 gpad.

| | | Hydraulic head, h | | |
|---|---|---|---|---|
| Type of Liner | Leakage mechanism | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| FML alone | Permeation<br>Small hole | 0.001<br>30 | 0.1<br>100 | 10<br>300 |
| | TOTAL | 30 | 100 | 300 |
| | Permeation<br>Standard hole | 0.001<br>300 | 0.1<br>1,000 | 10<br>3,000 |
| | TOTAL | 300 | 1,000 | 3,000 |
| Composite liner (good) contact) | Permeation<br>Standard hole | 0.001<br>0.02 | 0.1<br>0.2 | 10<br>3 |
| | TOTAL | 0.02 | 0.3 | 13 |
| Composite liner (poor) contact) | Permeation<br>Standard hole | 0.001<br>1 | 0.1<br>8 | 10<br>50 |
| | TOTAL | 1 | 8 | 60 |

Values of leakage rate in Ltd or gpad

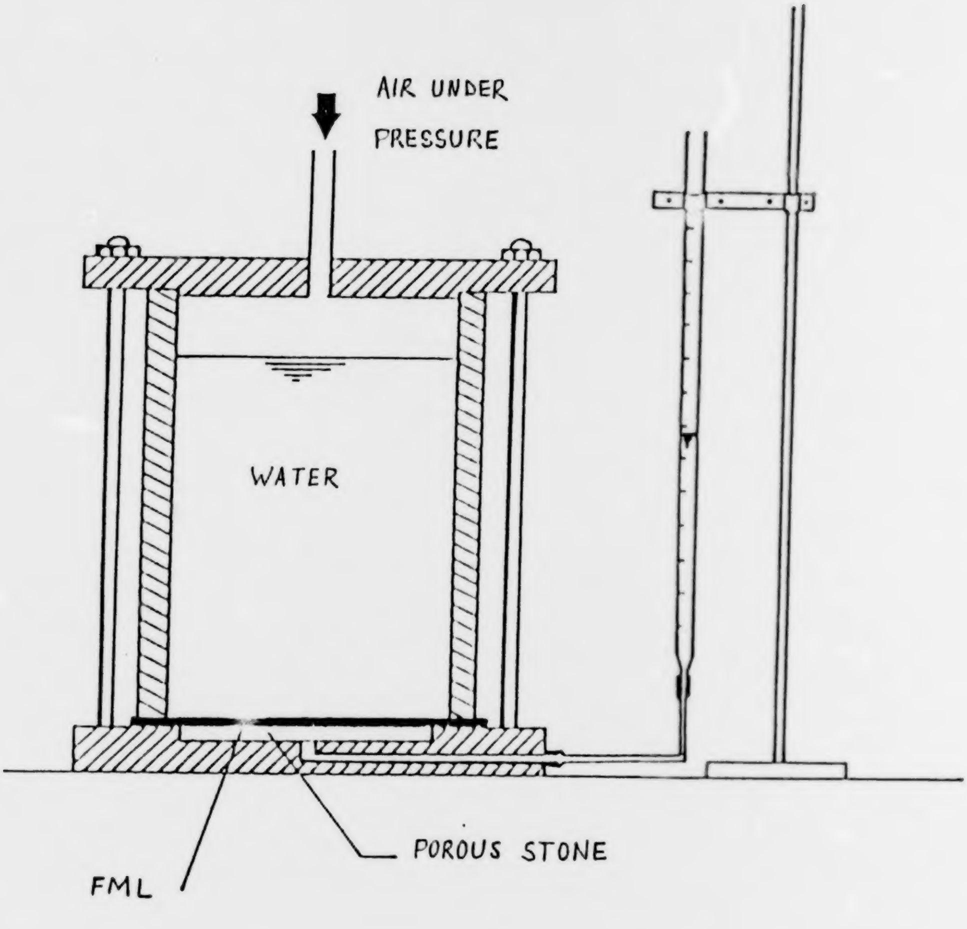

Figure 2.2-1. Permeameter used to evaluate flow through intact FMLs at the University of Grenoble (France).

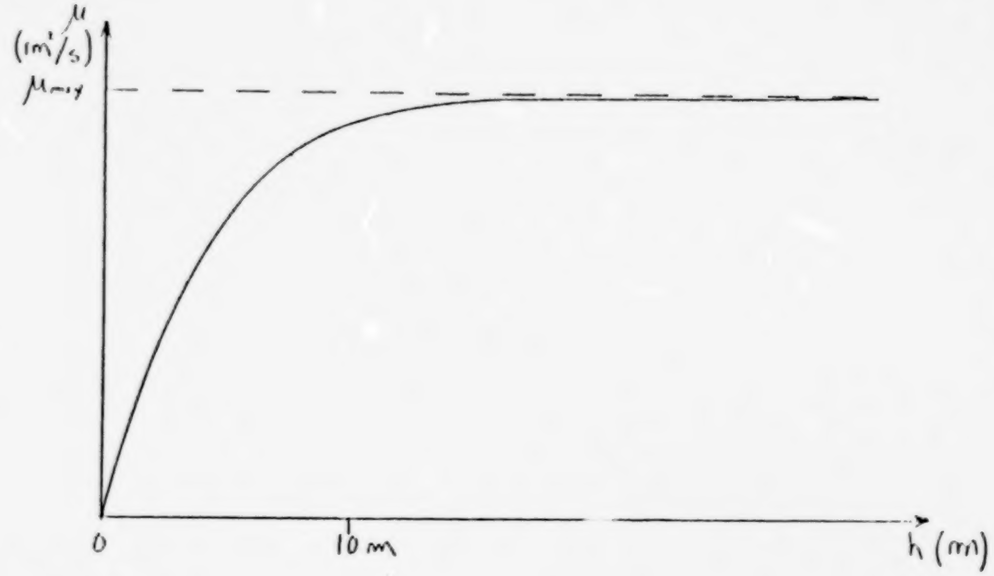

Figure 2.2-2. Typical shape of the curve giving the coefficient of migration, $\mu_g$, as a function of the hydraulic head, h.

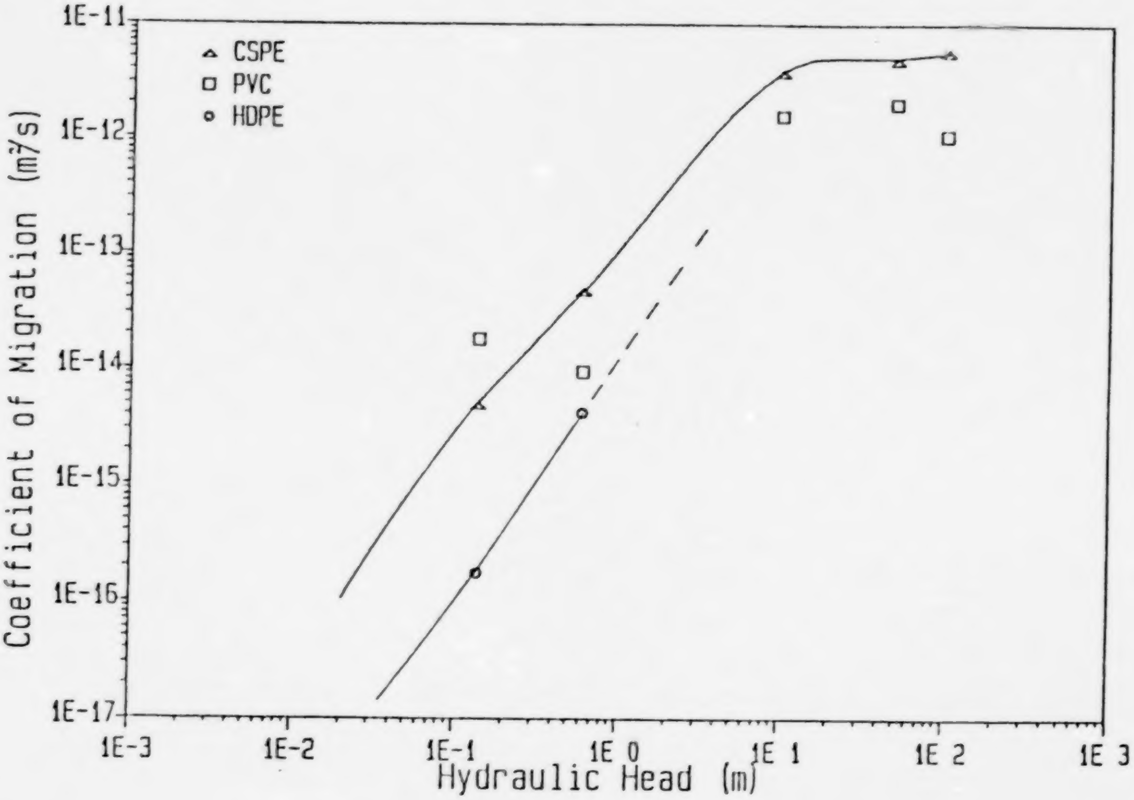

Figure 2.2-3. Values of coefficient of migration, $\mu_g$, for various FMLs from Table 2.2-6.

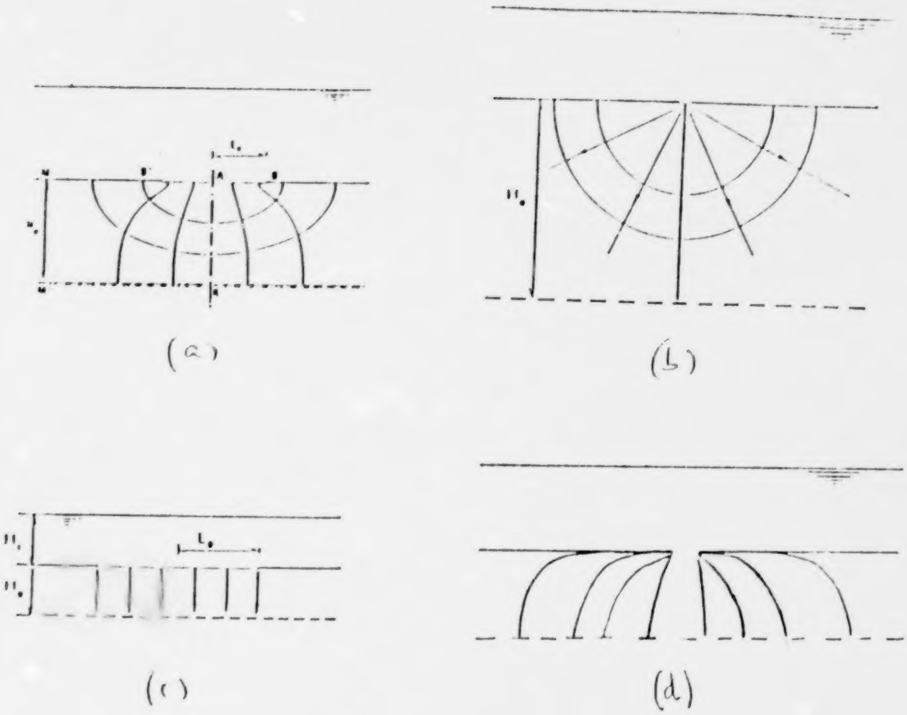

Figure 2.2-4. Flow nets for the four cases considered in two-dimensional theoretical studies related to leakage through composite liners due to hole in FML, assuming perfect contact between FML and soil layer: (a) entire soil layer saturated; (b) radial flow; (c) vertical flow; (d) actual flow. As demonstrated by Faure [1979], the actual flow is limited laterally by a phreatic surface. Note that in cases (a), (b), and (d), there is flow in the soil along the interface, although there is no flow between the FML and the soil because there is no space between the FML and the soil in the considered cases since perfect contact is assumed.

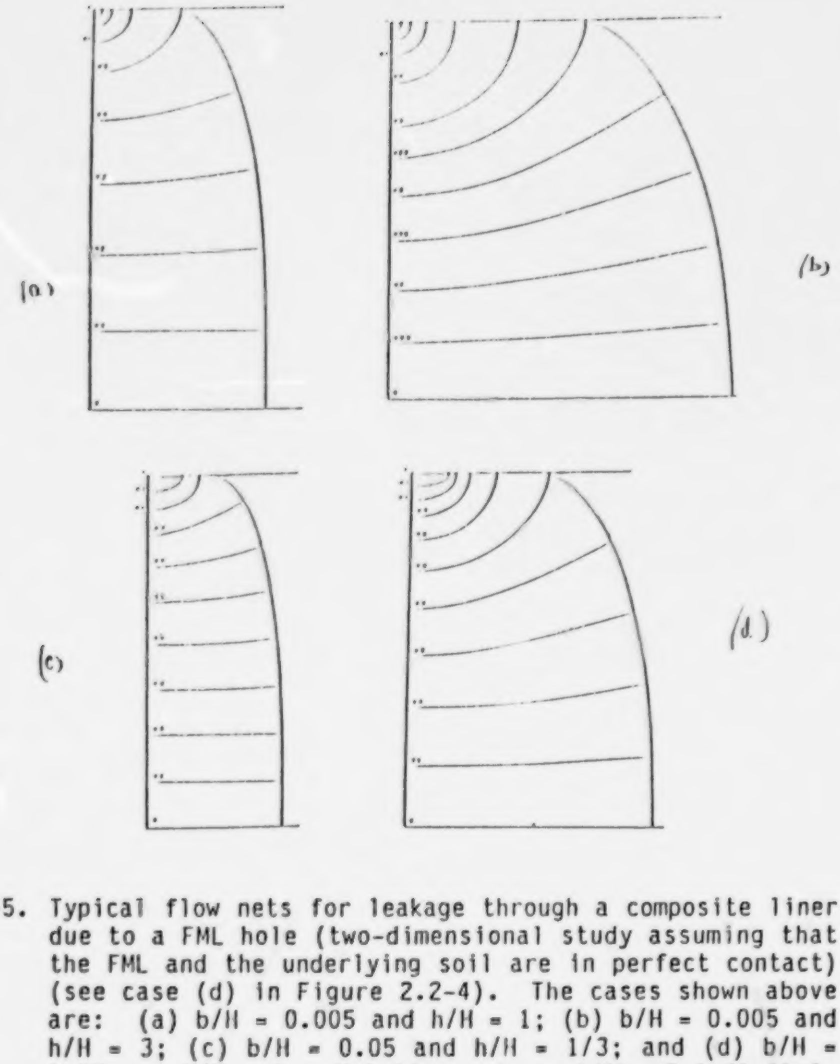

Figure 2.2-5. Typical flow nets for leakage through a composite liner due to a FML hole (two-dimensional study assuming that the FML and the underlying soil are in perfect contact) (see case (d) in Figure 2.2-4). The cases shown above are: (a) $b/H = 0.005$ and $h/H = 1$; (b) $b/H = 0.005$ and $h/H = 3$; (c) $b/H = 0.05$ and $h/H = 1/3$; and (d) $b/H = 0.05$ and $h/H = 1$. Notation: b = width of infinitely long hole (slot) in the FML; h = hydraulic head on top of the FML; and H = thickness of the soil layer underlying the FML [Faure, 1979].

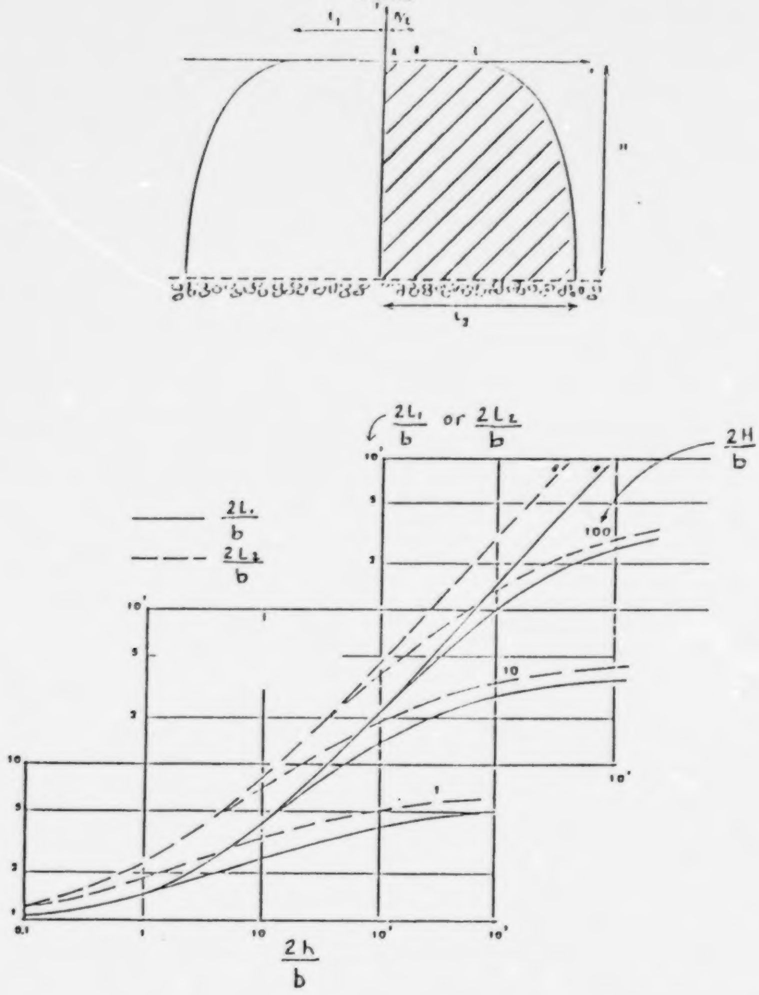

Figure 2.2-6. Lateral extent of the phreatic surface limiting the flow in the soil layer due to a hole in the FML. This chart is related to the two-dimensional case (the hole is a slot of width b) and perfect contact is assumed between the FML and the soil layer [Faure, 1979].

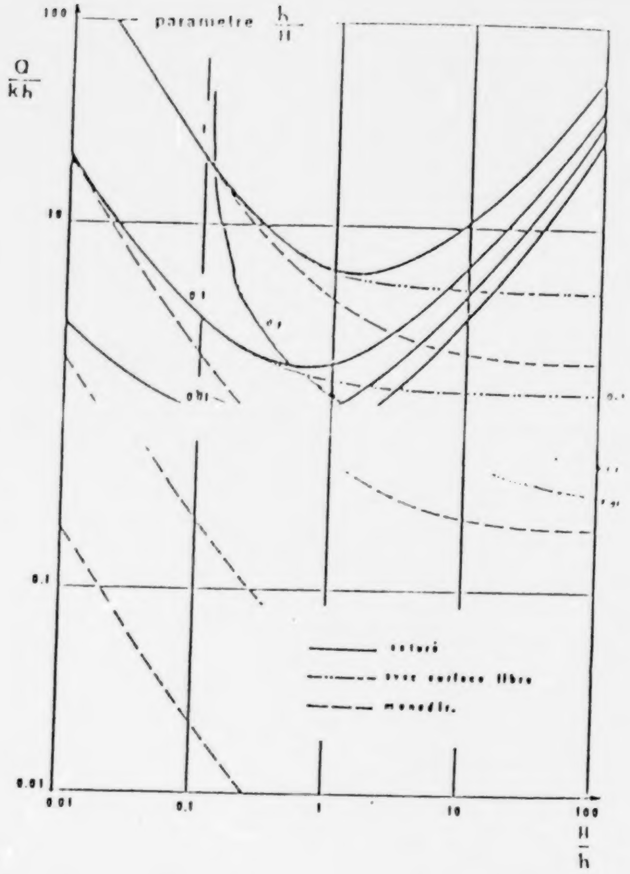

Figure 2.2-7. Leakage rates through a composite liner due to a slot of width b in the FML (two-dimensional case), assuming perfect contact between the FML and the soil. Calculations were made with several assumptions regarding flow: (a) soil entirely saturated by the flow; ($b_1$) radial flow using Equation 2.2-13; ($b_2$) radial flow using Equation 2.2-15; (c) vertical flow; (d) actual flow. Cases (a) through (d) are illustrated in Figure 2.2-4.

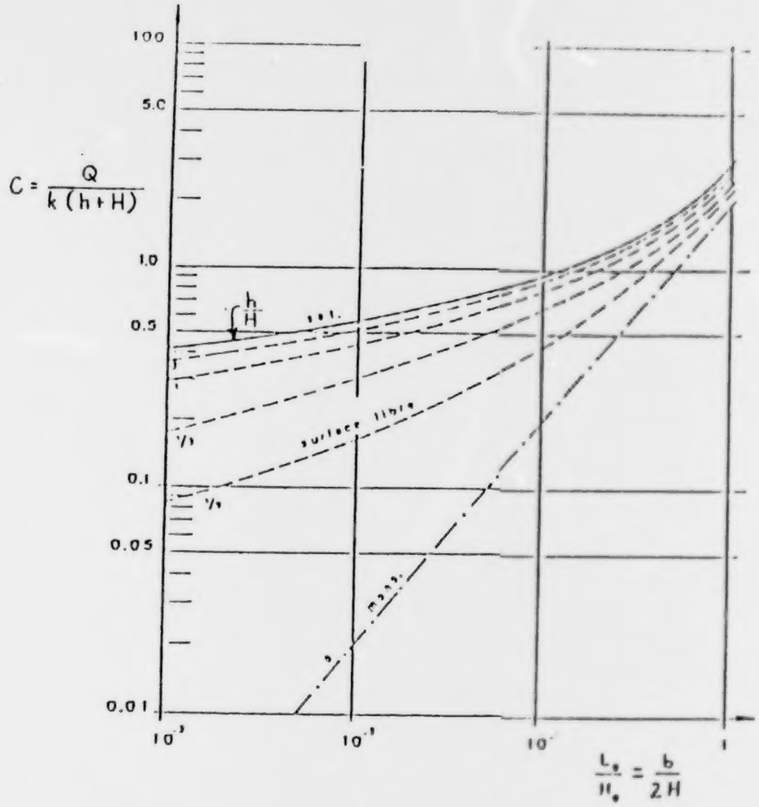

Figure 2.2-8. Chart giving dimensionless coefficient C to be used in Equation 2.2-14 which gives the leakage rate through a composite liner due to a slot in the FML (two-dimensional case). Coefficient C can also be used in Equation 2.2-22 to make an approximate evaluation of the leakage through a composite liner due to a circular hole in the FML (three-dimensional case). Notation: h = hydraulic head on top of the FML; b = width of the slot (to be replaced by the diameter d of a circular hole when the chart is used for the three-dimensional case); and H = thickness of soil layer.

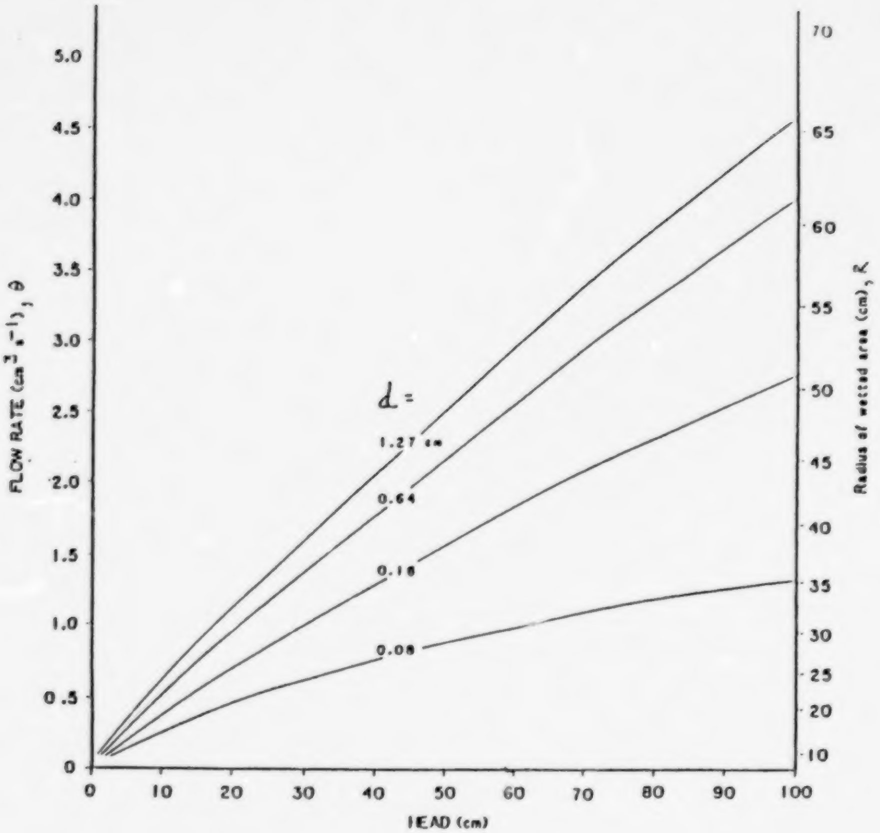

Figure 2.2-9. Leakage through a composite liner due to a hole in the FML [Brown et al.]. Chart giving the leakage rate, Q, and radius, R, of the wetted area as a function of the hydraulic head on the FML, for a compacted soil hydraulic conductivity $k_c = 3.4 \times 10^{-6}$ m/s ($3.4 \times 10^{-4}$ cm/s). Notation: d = diameter of the FML hole; and h = hydraulic head on the FML. Note: although the chart in [Brown et al.] is labeled "$k_c = 10^{-4}$ cm/s", it seems to us that it was established for $3.4 \times 10^{-4}$ cm/s.

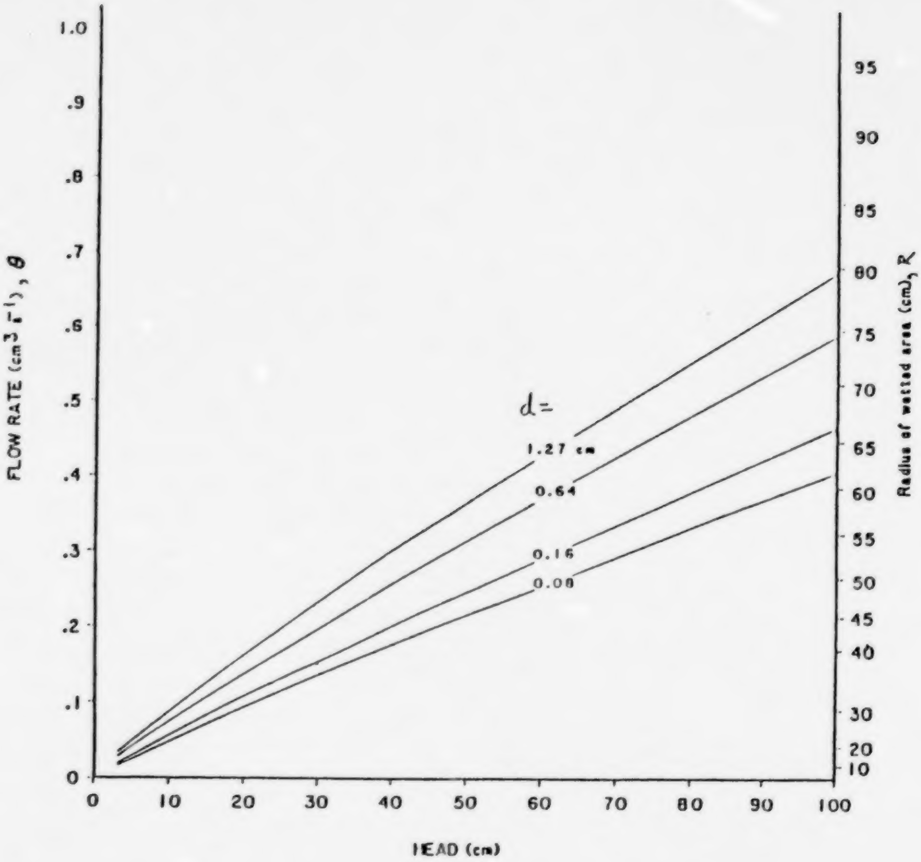

Figure 2.2-10. Leakage through a composite liner due to a hole in the FML [Brown et al.]. Chart giving the leakage rate, Q, and radius, R, of the wetted area as a function of the hydraulic head on the FML, for a compacted soil hydraulic conductivity $k_c$ = 3.4 x $10^{-7}$ m/s (3.4 x $10^{-5}$ cm/s). Notation: d = diameter of the FML hole; and h = hydraulic head on the FML. Note: although the chart in [Brown et al.] is labeled "$k_c$ = $10^{-5}$ cm/s", it seems to us that it was established for 3.4 x $10^{-5}$ cm/s.

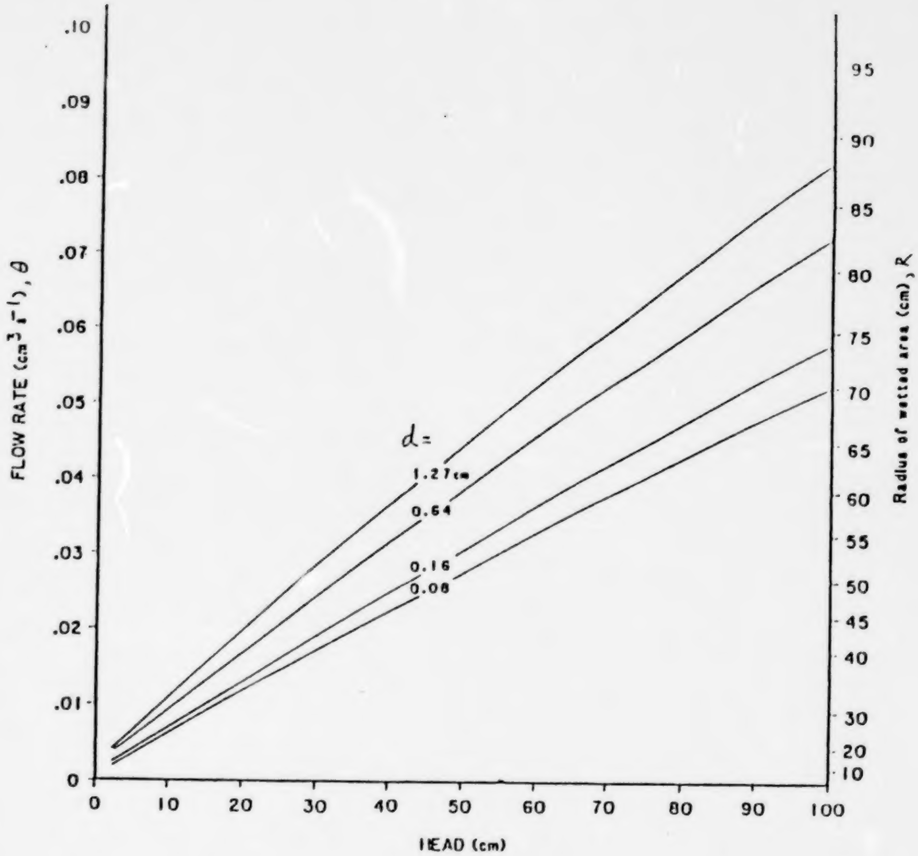

Figure 2.2-11. Leakage through a composite liner due to a hole in the FML [Brown et al.]. Chart giving the leakage rate, Q, and radius, R, of the wetted area as a function of the hydraulic head on the FML for a compacted soil hydraulic conductivity $k_c = 3.4 \times 10^{-8}$ m/s ($3.4 \times 10^{-6}$ cm/s). Notation: d = diameter of the FML hole; and h = hydraulic head on the FML. Note: although the chart in [Brown et al.] is labeled "$k_c = 10^{-6}$ cm/s", it seems to us that it was established for $3.4 \times 10^{-6}$ cm/s.

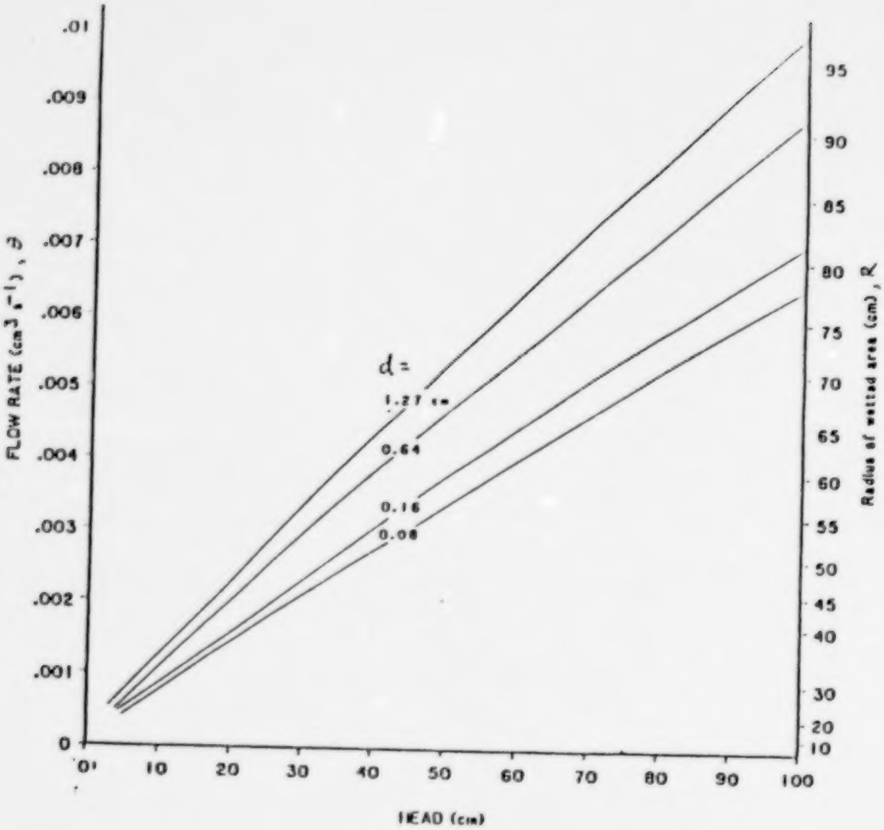

Figure 2.2-12. Leakage through a composite liner due to a hole in the FML [Brown et al.]. Chart giving the leakage rate, Q, and radius, R, of the wetted area as a function of the hydraulic head on the FML, for a compacted soil hydraulic conductivity $k_C = 3.4 \times 10^{-9}$ m/s ($3.4 \times 10^{-7}$ cm/s). Notation: d = diameter of the FML hole; and h = hydraulic head on the FML. Note: although the chart in [Brown et al.] is labeled "$k_C = 10^{-7}$ cm/s", it seems to us that it was established for $3.4 \times 10^{-7}$ cm/s.

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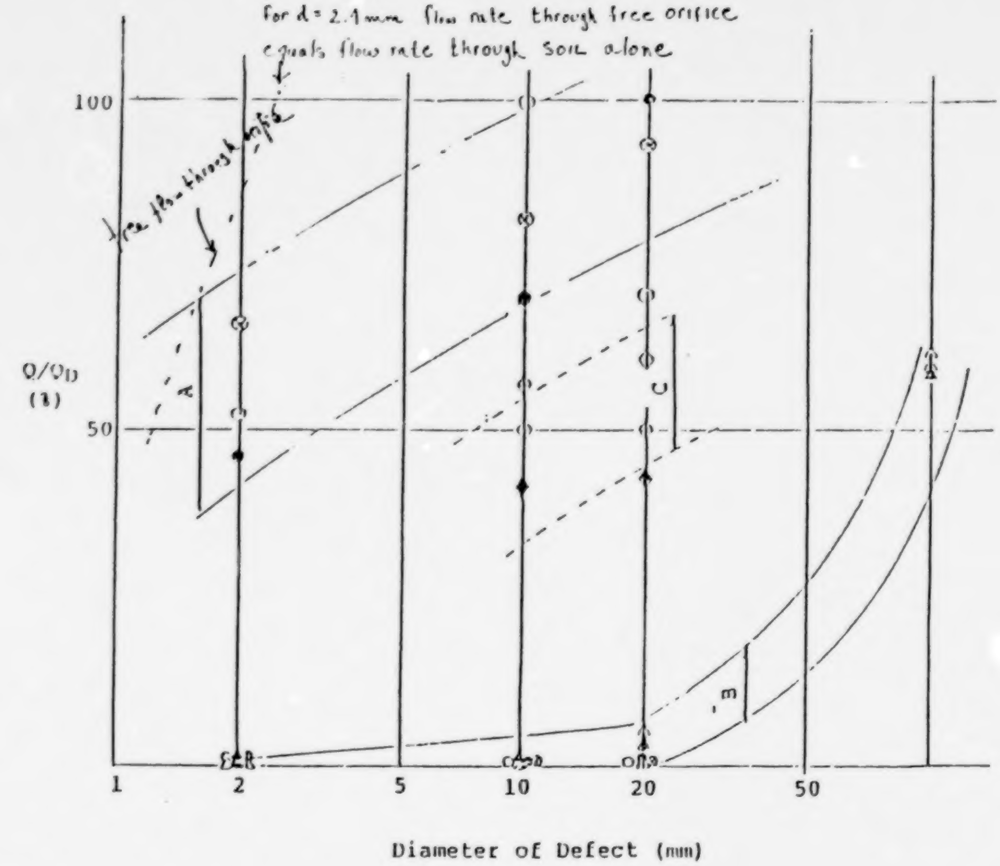


Figure 2.2-13. Leakage rates measured in tests conducted with a FML having a circular hole [Fukuoka, 1985]: (A) no soil cover on the FML, no geotextile between the FML and the soil; (B) there is a geotextile between the FML and the soil, but there is no soil cover on the FML; and (C) there is a soil cover on the FML and no geotextile between the FML and the soil. Notation: Q = leakage rate measured in the tests; and $Q_D$ = leakage rate when there is no FML (i.e., leakage rate governed by Darcy's flow through the soil).

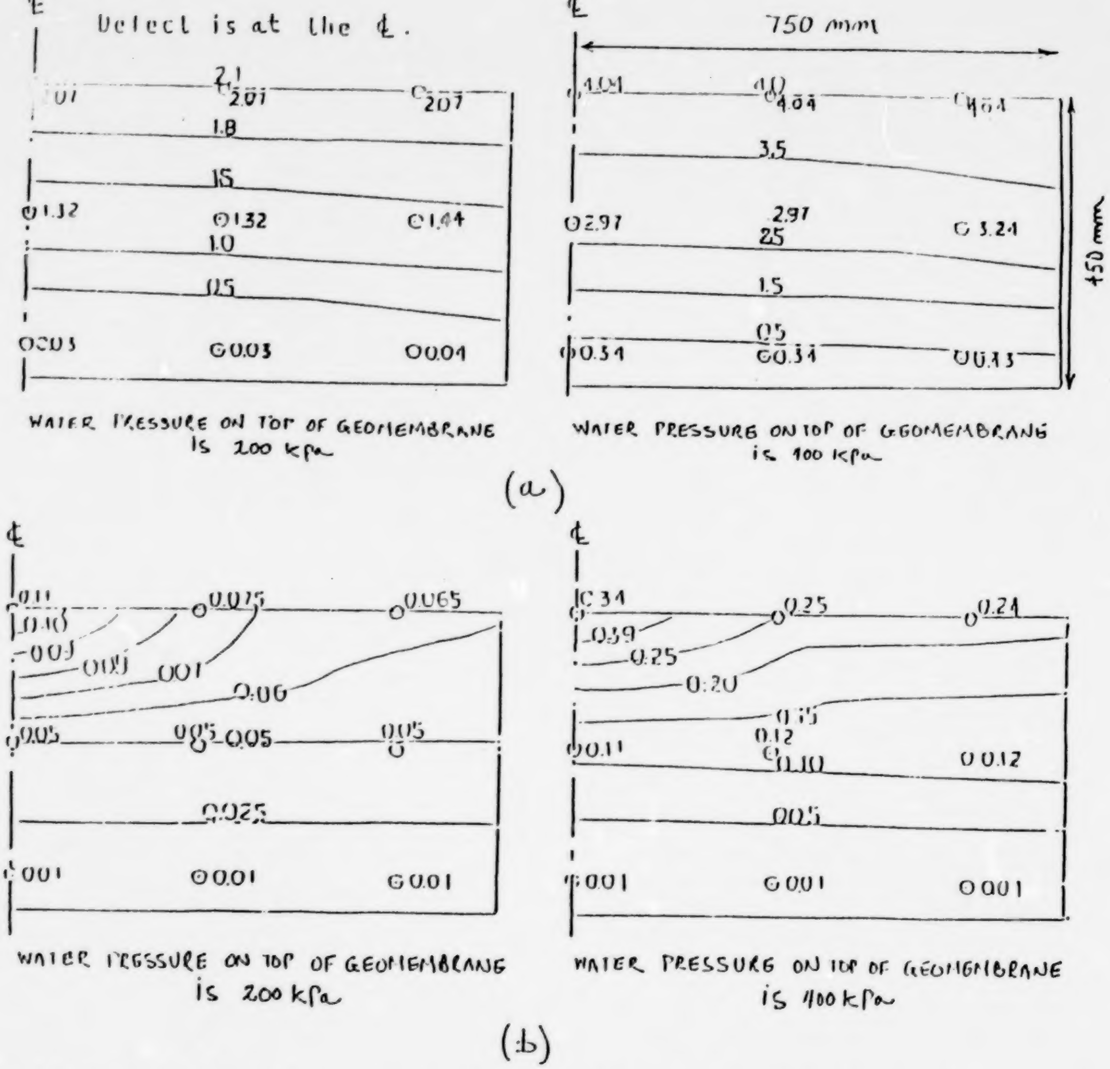
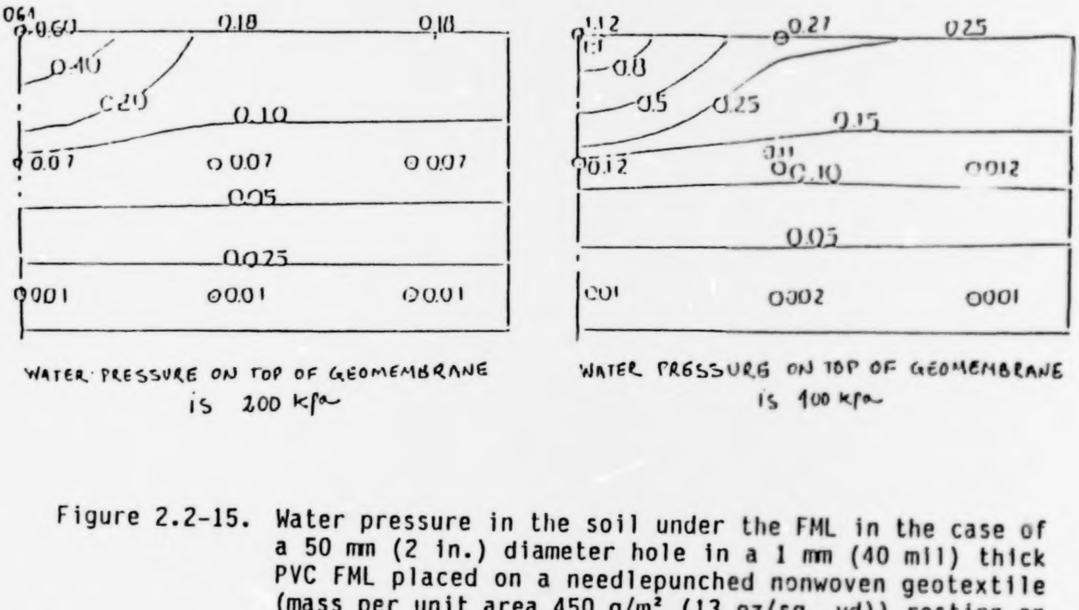


Figure 2.2-14. Water pressure in the soil under the FML in the case of a 20 mm (3/4 in.) diameter hole in a 1 mm (40 mil) thick PVC FML; (a) the FML is placed directly on the soil; and (b) there is a geotextile between the FML and the soil [Fukuoka, 1985].



WATER PRESSURE ON TOP OF GEOMEMBRANE IS 200 kPa

WATER PRESSURE ON TOP OF GEOMEMBRANE IS 100 kPa

Figure 2.2-15. Water pressure in the soil under the FML in the case of a 50 mm (2 in.) diameter hole in a 1 mm (40 mil) thick PVC FML placed on a needlepunched nonwoven geotextile (mass per unit area 450 g/m² (13 oz/sq. yd)) resting on the soil [Fukuoka, 1985].

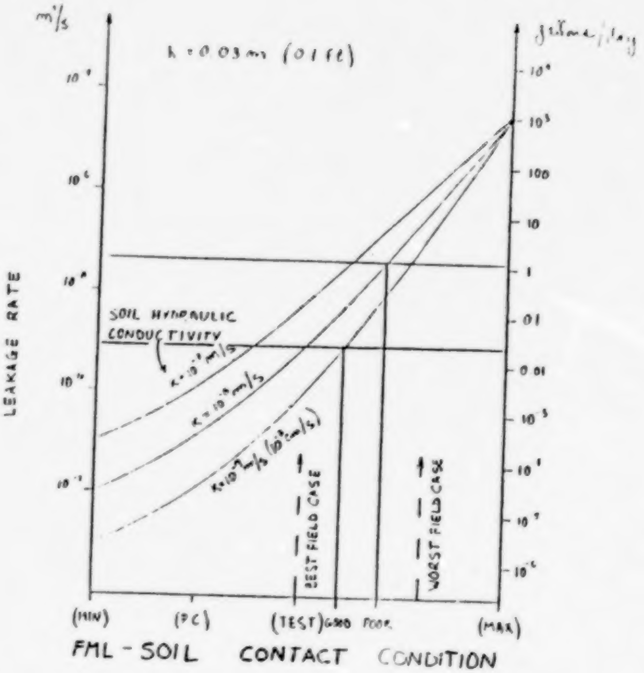


Figure 2.2-16. Graph giving the leakage rate in case of leakage through a FML hole in a composite liner. The hydraulic head is 30 mm (0.1 ft) and the hole area is 1 cm² (i.e. diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e., the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

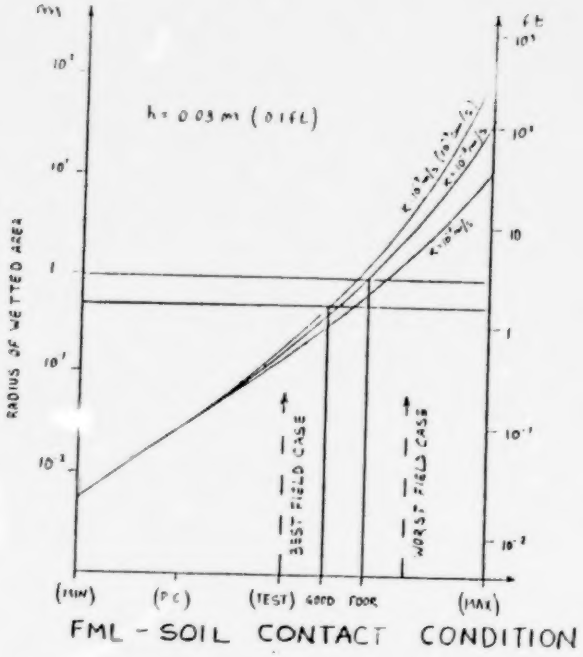


Figure 2.2-17. Graph giving the radius of the wetted area in case of leakage through a FML hole in a composite liner. The hydraulic head is 30 mm (0.1 ft) and the hole area is 1 cm² (i.e., diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e., the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

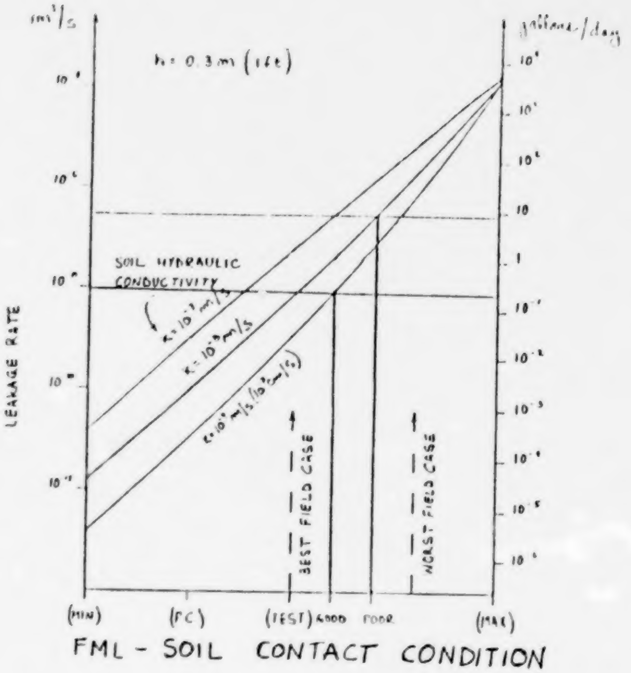


Figure 2.2-18. Graph giving the leakage rate in case of leakage through a FML hole in a composite liner. The hydraulic head is 0.3 m (1 ft) and the hole area is 1 cm² (i.e., diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e., the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

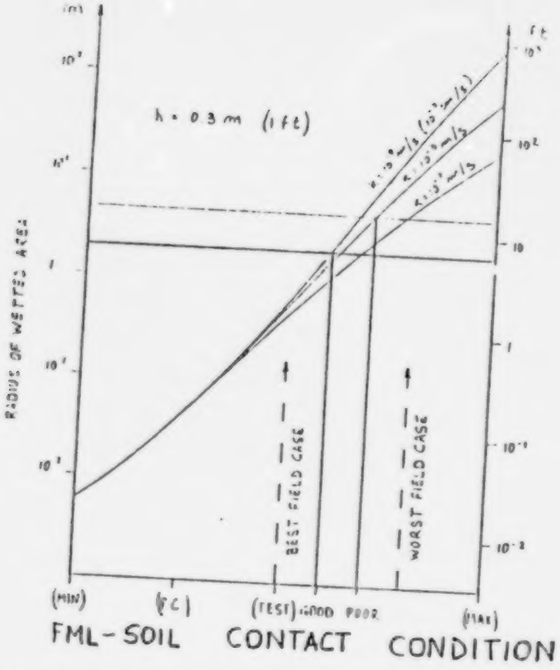


Figure 2.2-19. Graph giving the radius of the wetted area in case of leakage through a FML hole in a composite liner. The hydraulic head is 0.3 m (1 ft) and the hole area is 1 cm² (i.e., diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e., the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

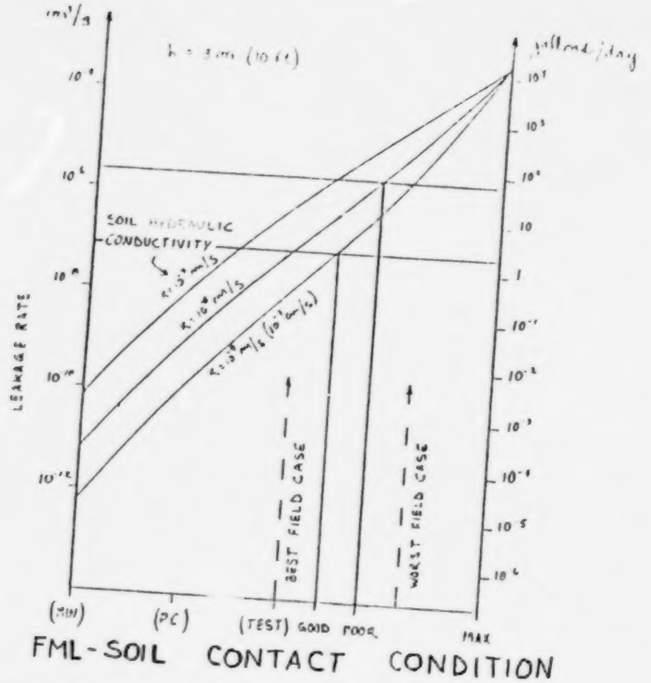


Figure 2.2-20. Graph giving the leakage rate in case of leakage through a FML hole in a composite liner. The hydraulic head is 3 m (10 ft) and the hole area is 1 cm² (i.e., diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e., the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

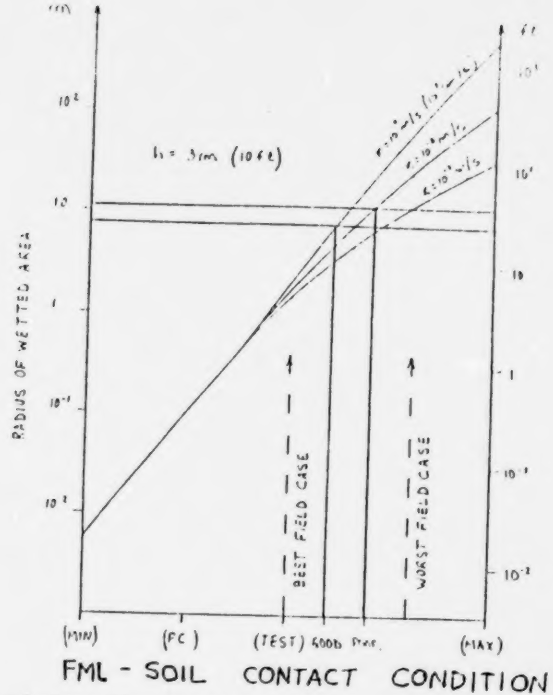


Figure 2.2-21. Graph giving the radius of the wetted area in case of leakage through a FML hole in a composite liner. The hydraulic head is 3 m (10 ft) and the hole area is 1 $cm^2$ (i.e., diameter of 11.3 mm). Because of uncertainties in the analytical analyses as well as the large influence of soil conditions and contact between the FML and the soil, only a range of values can be given. Field conditions can be anywhere between the two extremes: (1) best, i.e. the soil is well compacted, flat and smooth, has not been deformed by rutting during construction, and has no clods and cracks, and the FML is flexible and has no wrinkles; and (2) worst, i.e., the soil is poorly compacted, has an irregular surface and is cracked, and the FML is stiff and exhibits a pattern of large, connected wrinkles. Abbreviations: GOOD and POOR = good and poor field conditions; MIN, P.C., TEST, and MAX are defined in Table 2.2-11.

## 2.3 LEAK DETECTION TECHNOLOGIES

### 2.3.1 Review of Available Technologies

### 2.3.1.1 Introduction

The technologies available for leak detection at land disposal units fall into two categories: (1) drainage layer technologies using the leachate collection and removal system between the top and bottom liners as the leak detection system; and (2) innovative technologies involving the use of remote sensing techniques, hereafter referred to as geophysical leak detection systems.

The use of leachate collection and removal systems as leak detection systems is discussed in Section 2.3.1.2. Three geophysical methods are promising for use as leak detection systems and are described and evaluated in Section 2.3.1.3 through 2.3.1.5. These are electrical resistivity, time domain reflectometry, and acoustic emission monitoring. Other less developed innovative technologies are briefly discussed in Section 2.3.1.6.

### 2.3.1.2 Leachate Collection and Removal Systems

The concept of using the leachate collection and removal system (LCRS) between the top and bottom liners for leak detection is that by monitoring the liquids that accumulate in the LCRS sump, the presence of leaks can be detected. This method of leak detection has several attractive features. In addition to providing leak detection, the method provides information on the volume of leakage collected and, if the leakage is chemically analyzed, information on its chemical constituents. The LCRS technology is proven through in-field use and the necessary components are readily available. The components of the LCRS are also durable. It is a direct method of detection which does not require sophisticated data interpretation. In addition, since a LCRS between liners is already a statutory requirement of RCRA, no new component is added to the lining system.

### 2.3.1.2.1 Principles

A leachate collection and removal system between the top and bottom liners consists of a layer of drainage media designed and constructed to transmit flow laterally, between the liners, to collection points. The drainage layer can consist of either granular materials or synthetic materials. Detection is possible with such systems because the layer between the liners functions as a drain. To function as a drain, the layer between the liners must possess a much greater capability of transmitting flow than the bottom liner. This is possible only if the layer (drainage layer) is much more permeable than the bottom liner.

The mechanism by which lateral flow may occur in the drainage layer is as follows. Liquid entering the drainage layer will flow vertically (with some lateral wetting) until the less permeable bottom liner is encountered. Once encountered, the predominantly vertical flow will be retarded and liquid will begin to accumulate on the liner. Vertical flow will continue at a rate determined primarily by the hydraulic conductivity of the bottom liner. Lateral flow will begin when a threshold level of saturation above the bottom liner is reached such that the gravitational force on the liquid is greater than the capillary and adsorptive forces of the drainage layer media. If, on the other hand, the rate of infiltration into the bottom liner is faster than the rate of liquid accumulation above the bottom liner, then lateral flow will never occur. Flow in a porous drainage layer is governed by Darcy's equation for both saturated and unsaturated conditions [Todd, 1980]. Flow is a function of hydraulic transmissivity, hydraulic gradient and width of the cross-sectional area in the direction of flow [Figure 2.3-1]. Hydraulic transmissivity is the product of the in-plane hydraulic conductivity and the saturated thickness of the drainage layer. In-plane hydraulic conductivity is a property of the drainage material and the liquid. This property is typically evaluated under saturated flow conditions. Under unsaturated flow conditions, capillary and adsorptive forces restrict the movement of liquids in porous materials and therefore decrease hydraulic conductivity. The lower the degree of saturation, the lower the hydraulic conductivity of the material, other factors being constant.

The hydraulic gradient in a saturated drainage layer is a function of the slope of the layer and the pressure differential between the inlet and the outlet. Under gravity flow, the pressure differential is zero and the hydraulic gradient is due to the slope. Because the drainage layer is designed to be far more permeable than the liner above it and because landfills and waste piles are required to have a leachate collection system above the top liner, pressure flow would not be expected in the drainage layer except in the unusual circumstance of a substantial top liner failure in a surface impoundment.

### 2.3.1.2.2 Evaluation

In Section 2.1.2.4.3 it was stated that ideally a leak detection system should rapidly provide information on the rate of leakage.

An advantage of a LCRS type leak detection system (a LDCRS) is that it provides information on the rate of leakage. This is accomplished by monitoring the volume of liquids that collect in the LDCRS sumps. However, there is some quantity of leakage that will not be detected in the LDCRS sump as a result of: (1) capillary, adsorptive and frictional forces which restrict the movement of liquids in the porous materials of the drainage layer and at the surface of the bottom liner; and (2) permeability of the bottom liner. (The detection sensitivity of the LDCRS to small leaks increases as the hydraulic conductivity of the bottom liner decreases. Therefore composite bottom liners provide more sensitive leak detection than compacted soil bottom liners.) An advantage of the LDCRS arising from its ability to collect leakage is that information on chemical constituents in the leachate can be obtained through chemical analysis.

The rapidity with which a LDCRS will permit leak detection is reflected by the detection time calculated for the leak detection system. This LDCRS performance characteristic refers to the time between when leakage passes through the top liner into the LDCRS, and the time when it is available for detection in the LDCRS sump. For a given type of leak and rate of leakage, the detection time is primarily dependent on the hydraulic conductivity and thickness of the drainage media in the LDCRS, the capillary stresses in the drainage media, the slope of the LDCRS and the length of the drainage path. Through proper design and selection of materials, leak detection times in the range of a day can be achieved. (This represents BDAT for LCRS-type leak detection systems, as discussed in Section 2.7.) Detection of leakage within a few days or less of a leak occurrence provides a system with minimal potential for leakage from the land disposal unit prior to leak detection. On this basis, it is concluded that leak detection times of a few days satisfy the criterion that the leak detection system detect leaks rapidly. Design of the LDCRS to meet the performance standards described in this paragraph is discussed in Section 2.8.

The ability to locate a leak within a LDCRS is limited and depends on the design of the system. The location of a leak can be narrowed to a zone if the system is divided into separate collection and removal zones. For a given unit, the size of zones is usually dictated by design and economic factors and site conditions. However, even with zoning, it is clear that these systems have limited capability for precisely locating leaks. However, as noted in Section 2.1.2.4.3, leak location identification is usually not very important.

LDCRS are reusable because the occurrence of a leak does not physically damage the system. However, the sensitivity of the system for detecting multiple leaks in the same collection sump is less than the sensitivity for a single leak.

LDCRS are highly reliable systems that can be expected to function well over time with minimal maintenance. Their primary component (i.e., the drainage media) are comprised of relatively inert materials, such as clean silica sands or high density polyethylene (HDPE) synthetic drainage materials.

LDCRS can function with minimal maintenance and at the same time provide continuous monitoring of top liner leakage. Continuous monitoring of top liner leakage is inherent to the system because leakage entering the system is automatically transmitted by the drainage layer to the LDCRS sump for monitoring. Leakage may thus be monitored by only performing periodic measurements and records of liquids at the LDCRS sump.

LDCRS are capable of detecting leaks in all areas of the bottom and sidewalls of a waste management unit. This capability results from the consideration that drainage layers of LDCRS are required to be installed under all areas that are likely to be exposed to the waste or its leachate.

### 2.3.1.3 Electrical Resistivity

This section has been excerpted from GCA [1984b]. It has been reproduced here with modifications and additions.

Electrical resistivity is a geophysical technique whereby an electrical current is introduced into the ground by a pair of surface electrodes and the resultant potential field, as measured by a second pair of electrodes,is interpreted to detect anomalies (leakage).

Dry soil and rock materials are typically highly resistive (resist the flow of electric current). As the moisture content of soil or rock increases, the resistivity generally decreases significantly (i.e., the material becomes more conductive), and as various salts and free ions become incorporated into ground water, the resistivity decreases even further. Because of this phenomenon, resistivity has been used successfully to locate and map leachate contaminant plumes [Benson et al., 1982].

The potential application of the electrical resistivity technique to leak detection in FML-lined waste management units has been investigated by EPA at the Southwest Research Institute (SRI) [Schultz et al., 1984] using the pole-dipole resistivity technique. More recently, Foote Mineral Company [1986] has proposed a patented leak detection technique based on the dipole-dipole resistivity technique. The discussion that follows is based primarily on data from studies conducted at SRI. However, recent information from Foote indicates promise for Foote's ER technique as well.

#### 2.3.1.3.1 Principles

Because of their high electrical resistance, FMLs act as electrical insulators between the waste material above and the LDCRS below the FML. A failure in the FML establishes a path of low resistance which can be detected and located with measuring devices at the surface.

Resistivity measurements are taken using two current electrodes and two potential electrodes which may be placed in different configurations. The electrical resistivity (ER) method used by SRI to detect leaks from waste management units lined with FMLs utilizes the pole-dipole electrode configuration illustrated in Figure 2.3-2. A current source electrode is placed inside the waste area and a remote current electrode is placed outside of the area at a remote distance. This distance should be at least 50 times the distance between the two potential electrodes [Waller and Davis, 1982]. One of the potential electrodes is used as a reference point and remains stationary while the other potential electrode is positioned at preselected measuring points over the waste site. Theoretically, a fixed array of potential measurement electrodes could be installed where frequent or continuous monitoring is desired. However, a fixed array arrangement would require such sophisticated wiring and data interpretation systems that the practicality of the arrangement is questionable.

A direct current (DC) or low frequency alternating current (AC) injected at the source electrode flows to the remote electrode (Figure 2.3-3) The flow of current will be inhibited by the FML except over the upper edges of the liner and at leak points. Since surface potential measurements are affected by the current distribution, plots of equipotential lines will show distortions in the area of the leak. The use of computers for recording and processing data has greatly reduced the time required for data interpretation.

### 2.3.1.3.2 Recent Studies

Electrical resistivity (ER) techniques have not been used in permanent installations for the purpose of continuous leak detection at waste management units to date. All known commercial applications have been for temporary applications at surface impoundments.

ER techniques have been studied extensively by researchers at the Southwest Research Institute (SRI) under the sponsorship of EPA on a 1-acre test impoundment.

The ER system used by SRI is illustrated in Figure 2.3-4. The current source electrode, remote current electrode, and reference electrode remained stationary during the surveys. The potential measurement electrode was attached to a cable which was positioned over preselected measurement lines radiating from the current source electrode to the edge of the impoundment. The selected lines were 5 or 10 degrees apart. During actual measurements, a motorized winch reeled the electrode from the edge of the impoundment across the water surface back to the current source electrode. Figure 2.3-5 shows equipotential plots generated by SRI for (a) no leaks, and (b) a 0.3 m (1 ft) diameter leak in 1.5 m (5 ft) of water.

Testing of the ER method by SRI indicates that the technique is highly sensitive to leakage from surface impoundments. The presence of a 25 mm (1 in.) diameter leak was detected in the lined test impoundment. In addition, the position of the leak was located to within one foot of the actual location in 1.5 m (5 ft) of water.

Several other experiments were conducted at SRI. They demonstrated the following capabilities of the ER system:

- An increase in water depth from 1.5 to 2.0 m (5.0 to 6.5 ft) reduced the surface potential voltages near the leak but did not hinder data interpretation.
- The distortion of equipotential lines produced by a 9.1 m (30 ft) long simulated seam leak was readily apparent and significantly different from equipotential lines produced by a point source leak.
- The locations of three simultaneous leaks ranging in size from 2.5 to 30 cm (1 to 12 in.) were identified.

### 2.3.1.3.3 Evaluation

The field experiments conducted at SRI demonstrated the potential of the ER system for leak detection at surface impoundments lined with a single FML liner. The potential for applying this technique to actual hazardous waste management units is evaluated hereafter.

In Section 2.1.4.2.3 it was noted that the important performance requirements for leak detection systems are that they evaluate the leakage rate and provide the information rapidly. With ER techniques, it is not possible to evaluate the leakage rate. This is a drawback of ER-type systems. ER techniques can be used to detect leakage rapidly if the ER system is used as a continuous monitoring system or if ER surveys are done frequently. With only portable equipment however, ER surveys are time consuming and expensive. An advantage of ER techniques is that they can be used to determine leak location. This is especially advantageous in surface impoundments (see Section 2.1.4.2.3).

With respect to leak location, SRI researchers demonstrated that the ER systems can be sensitive to a 2.5 cm (1 in.) diameter leak in a 0.4 hectare (1 acre) impoundment, and that the leak can be located accurately to within 30 cm (1 ft). A 2.5 cm (1 in.) diameter leak is the smallest leak simulated in the SRI tests and is not necessarily the smallest leak the system is capable of detecting. However, it is clear that liquid transported through the liner as a vapor would not be detected by the ER technique because no current path would be produced by the vapor.

While the results of the research at SRI are impressive, there are a number of factors which could be expected to interfere with the performance of the system under routine use. These are addressed in the following paragraphs.

The electrical resistance properties of the liner materials used are of critical importance to the performance of the ER method. The primary liner must be composed of a highly resistant material such as an FML. The method cannot be used to detect leaks from an impoundment with a single or top liner consisting of compacted soil alone due to its comparatively low electrical resistance. In double liner systems, the composition of the drainage layer and the bottom liner are also of importance. Dry sand, gravel, and synthetic drainage materials all exhibit relatively high electrical resistance. When wetted, the resistance is lowered significantly. But current will flow to the remote current electrode along the leak path only if the bottom liner has a low electrical resistance (e.g., consists of compacted soil alone) or if the bottom liner is also leaking. A compacted soil bottom liner would allow the leak current path to be established while an intact composite bottom liner would not. Therefore, ER is not applicable to facilities with a composite bottom liner.

The application of the ER technique to landfills has not been demonstrated on a large scale. SRI tested the method on a 3 m by 3m (10 ft. by 10 ft.) scale model which was lined with a 0.15 mm (6-mil) thick black polyethylene sheet and filled with soil. Soil depth was varied from 7 to 14 cm (2.75 to 5.5 in.), and simulated leaks 15 cm (0.5 ft) in diameter were accurately detected and located [Schultz et al., 1984]. This experiment demonstrated that the ER technique could potentially be used for detecting leaks in a homogeneous, solid material.

The performance of resistivity techniques is affected by interferences such as buried metal pipes, metal fences, railroad tracks and stray currents from power lines. Highly conductive waste materials have the potential for limiting the effectiveness of the ER technique at waste management units [Waller and Davis, 1982]. Some limited experiments have been conducted at SRI with highly conductive metallic objects placed at various locations within the 3 m (10 ft) scale model filled with water. Their studies suggest that the metallic objects have no significant influence on the distribution of equipotential values. More research is needed in this area to assess the effects on equipotential measurements from heterogeneous deposits as will probably occur in any landfill.

The ER leak detection system used at SRI is not a continuous monitoring system but rather a survey technique. The equipment must be set up and operated, and potential measurements must be interpreted each time leak information is sought. Detection time, therefore, is a function of the technique but, more importantly, is a function of how often the technique is used.

SRI reports that it takes 8 hours to set up equipment, survey, and analyze the results from the one-acre test impoundment [ENR, 1984]. While the set-up time would remain fairly constant as the unit size increased, survey and data interpretation time would be expected to increase proportionally to size.

At some hazardous waste management units, frequent or continuous leakage monitoring may be necessary. An ER leak detection system could theoretically be installed permanently at such units by replacing the mobile potential measurement electrode with a set of stationary potential measurement electrodes. Equipotential plots would be constructed from the potential readings at each electrode. The resolution of the system would be a function of the spacing of measurement electrodes. While the cost of equipment for a continuous monitoring system would be sharply higher than for a portable system due to the need for multiple stationary potential electrodes, time for detection would be reduced and labor associated with mobilization and demobilization would be eliminated. Data interpretation, on a unit-cost basis, would be unchanged.

The ER leak detection system does not require the installation of below-ground equipment at the hazardous waste management unit. The electronic equipment needed, whether portable or permanently installed, is easily accessible for inspection and maintenance purposes.

The current source, reference potential, and potential measurement electrodes all come into direct contact with waste material. The material used in these electrodes must be resistant to chemical degradation and at the same time possess the required electrical properties. In the SRI study, brass was selected for the current source electrode, and copper for the reference potential electrode.

The reliability of the ER method of leak detection, as demonstrated at the test site, is high, provided interferences are minimal, electronic equipment is functioning properly, operators follow prescribed procedures, and data interpretation is performed by trained personnel. These conditions can be met with the implementation of a strict program of quality assurance/quality control.

### 2.3.1.4 Time Domain Reflectometry

This section has been excerpted from GCA [1984b]. It has been reproduced here with modifications and additions.

Historically, the Time Domain Reflectometry (TDR) technique has been used in the communications industry to locate faults in long transmission lines. Recently, TDR has been used to map soil moisture content along buried electrical transmission lines. The latter application has shown promise for early detection of liner failures at hazardous waste landfills and surface impoundments and has been the subject of a number of recent studies [Davis et al., 1984, 1983a, 1983b; Huck, 1982; Waller and Davis, 1982]. None of these studies has been conducted with actual waste containment sites.

### 2.3.1.4.1 Principles

The TDR technique measures the electrical property variations in soil along a pair of parallel transmission line conductors (Figure 2.3-6). A high frequency step-pulse is introduced by the TDR system which sets up both an electric field and a magnetic field between the two conductors. The soil in the zone between the two conductors thus becomes part of the transmission pathway.

The electrical properties of soils are different from water. The dielectric constant for dry soil ranges from 2 to 5, while that of water is about 80. Because of this rather large contrast, the TDR system is sensitive to soil moisture content, and is relatively insensitive to soil type. These properties make the TDR system potentially attractive for use under FML liners installed at hazardous waste management units (Figure 2.3-7).

### 2.3.1.4.2 Recent Studies

Several EPA-funded studies have been conducted to test the efficacy of TDR as a leak detection technique. These studies have ranged from scale-model laboratory studies to limited field studies on a model waste impoundment site [Davis et al., 1984, 1983a, 1983b; Huck, 1982]. Results have shown that leak detection by TDR is technically feasible with presently-available instrumentation. Transmission lines have been tested which varied from 2.0 to 20 m (6.6 to 65.6 ft) in length although theoretically it is possible to monitor up to 40 m (131 ft) of cable with present instrumentation. Transmission line separation in these experiments varied from 0.1 to 2.0 m (0.3 to 6.6 ft).

Some of the EPA findings include:

- Dry sandy soils (as would be present in a drainage layer) have dielectric properties similar to air and electromagnetic energy losses are not significant. Thus, these materials are suitable for use as a host material beneath an FML liner. The host material thickness should be at least that of the transmission line spacing;

- The ability of a TDR system to measure changes in dielectric properties of the host material is decreased as the spacing between the transmission line conductors is increased;

- The minimum detectable leak size that can be detected increases with increasing transmission line conductor spacing; and

- Present technology would permit the design of a dedicated TDR leak detection system along 1000 m (328 ft) transmission lines capable of detecting a 0.1 $m^2$ (1.0 $ft^2$) leak, although such resolution would be difficult.

### 2.3.1.4.3 Evaluation

The TDR technique tested showed promise in allowing accurate definition of the location and approximate size of a leak in a timely manner. Present TDR technology enables the design of a leak detection system that will detect and locate a leak in an area of 0.1 $m^2$ (1 $ft^2$). This would require a conductor separation no larger than 0.6 m (2 ft.), although 0.3 m (1 ft.) would be better. Detection of a leak of this size is subject to certain assumptions concerning the dielectric properties of the dry host material and those same properties when saturated by the leachate or impounded liquid, as discussed later in this section.

If a TDR leak detection system is monitored, it will have a rapid response time which may be on the order of minutes in the event of top liner failure, providing the leak is large enough to be detected. However, because monitoring is expensive, monitoring will typically be much less frequent.

The most critical aspects to be considered in the design of a TDR system are the physical and electrical properties of the granular drainage media and the geometry and spacing of the transmission line conductors. Each site will be different, depending on the size, materials available, and specific performance criteria required of the leak detection system employed.

The designer must know the minimum size of leak which must be detected to design a TDR leak detection system. The transmission line conductor separation must be no greater than twice the diameter of the "design leak". Sufficient contrast in the dielectric constant must exist between the drainage media hosting the transmission line conductors and the leachate or liquid to be detected by the TDR technique. In addition, the granular drainage layer must be at least as thick as the separation between transmission line conductors. For this reason, TDR cannot be used with synthetic drainage layers.

The water content of the drainage media when installed must be sufficiently low so that an adequate contrast in moisture content exists once a leak develops. During the model waste impoundment study conducted by EPA [Davis et al., 1983a] the sand drainage media in the test section had to be dried three times over a period of several months before they were placed.

Water content considerations underline the importance of selecting the proper materials for the drainage layer. A material with a low specific retention is necessary to avoid costly drying operations. Specific retention is defined as the ratio (expressed as a percentage) of the volume of water a material will retain after it has been saturated and allowed to drain. A coarse sand with a porosity of 40 percent may have a specific retention of 5 percent. A silty coarse sand may have a specific retention of 15 percent or greater.

Low specific retention materials are also desirable for reusability. If a leak develops in the primary liner of a surface impoundment, a TDR system may become functional again after the leak has been repaired and most or all of the leakage has drained by gravity.

The drainage layer should be made up of a well-compacted medium- to fine-grained sand with sufficient fines to horizontally disperse the wetting front of a leak thus increasing the TDR response [Davis et al., 1983a]. Horizontal dispersion of the wetting front will produce a wetted zone much larger in horizontal cross-sectional area than the actual leak. The drainage layer should not contain too many fines, however, as silts and clays rapidly attenuate the TDR signal. The apparent dilemma created by a simultaneous need for low specific retention and a good TDR signal transmission on the one hand, and horizontal dispersion on the other points out the need for further study. In addition, horizontal dispersion reduces leakage collection at the LCRS sump.

The transmission line conductors should be approximately 2.5 mm (0.1 in.) diameter stranded copper [Davis et al., 1983a]. Wires of this diameter are not easily damaged during installation and have performed well in this application. However, the potential for wire corrosion is uncertain. Durability is, therefore, a concern with this method.

Data acquisition would take place at an interval determined by the nature of the impounded waste materials and the potential impacts involved in impoundment failure. Design of a continuously monitored TDR system is feasible, but expensive. Such a system might involve manual or automatic switching from one transmission line pair to the next so that a central monitoring device could access information from any portion of the site. Newly-acquired data from each transmission line could be compared (manually or by computer) with an earlier data set from the same transmission line. Any differences between the earlier and new data sets would be evaluated in the context of possible top liner failure.

### 2.3.1.5 Acoustic Emissions Monitoring

This section has been excerpted from GCA [1984b]. It has been reproduced here with modifications and additions.

Acoustic emission monitoring (AEM) is based on the concept of detecting vibrations (acoustic emissions) produced by leaking liquids through the use of highly sensitive piezoelectric sensors (transducers). The potential of the technique as a means of detecting and locating leakage beneath FML liners was investigated following successful attempts to locate turbulent flow and seepage through earth dams. [Davis et al., 1983a; Koerner et al., 1984].

To date, the technology has not been proven at a full scale containment site, although installations of AEM systems have been planned at several of such sites.

### 2.3.1.5.1 Principles

Acoustic emission monitoring relies on the fact that turbulent flow with a velocity greater than 1 mm/s (0.04 in./s) through soil creates low frequency acoustic vibrations that can be detected and monitored [Koerner et al., 1984; Davis et al., 1983b].

Monitoring equipment consists of a highly sensitive transducer (a microphone or an undamped accelerometer) connected to an amplifier with an adjustment band-pass filter which provide a signal suitable for subsequent interpretation. Interpretation may be done manually by listening to an audible output or viewing a visual display, or electronically by an electronic spectrum analyzer. Portable single-channel field equipment is available which permits use of earphones to monitor acoustic emissions. This equipment also provides a strip chart record of monitored events. Output connectors are provided for additional recording, analysis, and display devices. Interpretation could be done by automated, continuous monitoring of multiple sensors at large scale installations.

### 2.3.1.5.2 Recent Studies

The potential of AEM for leak detection at lined containment sites has been evaluated by various researchers [Koerner et al., 1981, 1984; Waller and Davis, 1982; Huck, 1982; Davis et al., 1983a, 1983b]. Koerner et al. [1981 and 1984] conducted a series of experiments which investigated the use of different sensor configurations as well as the effect of flow rate on acoustic emissions and their characteristics. They observed that increased flow rates through sand and gravel media increased acoustic emissions and that these emissions showed an initial peak and then decreased to less than a threshold value after several minutes. In addition, they found that use of turbid water (38 g/liter clayey silt) increased the sensitivity to flow detection. Sensors located directly beneath a FML liner proved to be effective. Koerner et al. reasoned, however, that the cost of the large number of sensors required for a full-scale installation would be prohibitive and investigated the use of waveguides for transmission of raw signals to the sensor.

Davis et al., [1983a, 1983b] investigated many of the variables believed to have possible influence on the effectiveness of AEM for leak detection at containment sites. Davis confirmed the generation of acoustic emissions by liquid flow through sand and gravel media and concluded that the sounds have significant amplitude at frequencies up to 500 Hz and peak amplitude in the range of 100 to 200 Hz. The work also showed that sound amplitude increased with liquid velocity and with increased variation in soil grain size. However, sound amplitudes were observed to decrease to background levels after about one minute of water flowing through soil. The sound was believed to have been caused by collisions between soil particles and by air bubbles moving through the soil. Davis et al. further concluded that water alone flowing through soil does not produce significant sound energy. For sounds to be produced the turbulent movement of water mixed with fine grained soil or air through soil is required. The range of the sensors used in these experiments was about 1 meter. Davis concluded, as did Koerner, that emplacement of individual sensors at a containment site of four hectares (10 acres) would be impractical.

### 2.3.1.5.3 Evaluation

Laboratory and controlled field experiments have demonstrated the ability of AEM to detect leakage through FML liners under specified conditions. Proper spacing of sensors offers the potential to detect and locate leaks with a resolution of ±1 m (3.3 ft) in the case of individual sensors and with a resolution of ±0.3 m (1 ft) in the case of wire waveguides laid out in a grid. However, this technology has not been proven at a full scale installation.

AEM sensing equipment appears compatible for installation between double liners at newly constructed containment sites. Either individual sensors or wire waveguides could be installed in a granular drainage layer. Sensor leads would then be routed between liners to the ground surface at the edge of the containment basin for subsequent connection to monitoring and interpretation equipment. Figure 2.3-8 shows the installation of an AEM sensor in the granular drainage layer between two FMLs. Because of their size (current design is about 23 mm (0.9 in.) diameter X 200 mm (7.9 in.) length), individual sensors probably cannot be placed between FML liners separated only by a synthetic drainage layer. A wire waveguide grid may be more suitable for installation in this instance. The waveguides would be equipped with sensors at one end for connection to monitoring and processing equipment. Use of wire waveguides would have several advantages over individual sensors:

- Greater resolution in locating a leak may be obtainable.
- The number of sensing devices required may be reduced significantly. Thus capital costs may be reduced accordingly.
- Use of wire waveguides would permit locating sensors should a malfunction occur, in contrast to the inaccessibility of embedded sensors.
- Wire waveguides may be used with synthetic drainage layers located between two FMLs.

A number of drawbacks exist however, with the use of AEM in hazardous waste management units. These are:

- Individual sensors and leads buried between two liners must retain functional ability without repair during the units' active life and the 30 year post-closure care period. Should a malfunction occur in one sensor, repair would be very difficult. The use of wire waveguides with serviceable sensors reduces this concern somewhat; however, selection of waveguide material would still need to be done with full consideration of the potential for corrosion.
- AEM appears limited to the detection of leaks which result in turbulent flow and collisions between soil particles. Small leaks that increase in size gradually, and low velocity seepage

are likely to be undetected. In addition, the leak must be detected within a few minutes of occurrence before sound intensity diminishes to threshold values. This would necessitate design of an "on-line" continuous monitoring system to reduce the likelihood of missing a leak. High speed automatic electronic switching and processing would be a necessity, especially at large facilities.

- AEM equipment is sensitive to site background noise. Sensors placed at the end of wire waveguides would be susceptible to airborne as well as seismic noise. Individual sensors placed between liners would be protected from airborne noise sources; but may be sensitive to noises caused by nearby equipment or machinery.

#### 2.3.1.6 Other Leak Detection Technologies

This section has been excerpted from GCA [1984b]. It has been reproduced here with modifications and additions.

Leak detection technologies other than those described in the previous sections are not sufficiently developed to be considered as a primary alternative for leak detection in hazardous waste management units. These technologies may be useful as a confirmatory or backup techniques. A brief description follows on the applicability of each of these technologies.

##### 2.3.1.6.1 Lysimeters

Lysimeters are used to collect liquid samples from the zone of aeration or vadose zone in soils to monitor for the presence of leachate plumes. Monitoring within the vadose zone is desirable because it allows early detection of contaminants before significant contamination to soil and groundwater has occurred.

These are two common types of lysimeters: pressure-vacuum or suction lysimeters, and collection or trench lysimeters. Experience with both pressure-vacuum [Parizek and Lane, 1970; Apgar and Langmuir, 1971; Gerhardt, 1977; Johnson and Cartwright, 1980; Morrison and Ross, 1978] and collection lysimeters [Kmet and Lindorff, 1983] for leak detection at containment sites has been reported.

Pressure-vacuum lysimeters (Figure 2.3-9a) consist of a porous cup or sleeve attached to a small receiver vessel. The lysimeter is installed at the location and depth where a sample of soil water is desired (typically in a vertical borehole). The lysimeter is connected to the ground surface by two small diameter tubes. Sample collection is achieved by applying a vacuum to the lysimeter through one of the access tubes. Soil moisture held in tension then moves along the pressure gradient created by the vacuum, through the porous cup, and collects inside the lysimeter. The sample is transferred to the surface by releasing the vacuum and applying pressure to one of the access tubes. The water sample is forced to the surface through the other tube.

Pressure-vacuum lysimeters have a history of problems with clogging [Waller and Davis, 1982]. In addition, the porous ceramic cup typical of many lysimeters has been shown to affect sample integrity [Hanson and Harris, 1975]. It is now possible to purchase pressure-vacuum lysimeters manufactured entirely from PVC or Teflon. This should reduce or eliminate sample alteration problems associated with ceramic cups. In addition, greater strength and a lesser tendency to clog is reported for the Teflon cups [Castro and Timmons, 1983]. To reduce or prevent clogging, pressure-vacuum lysimeters are packed with fine silica sand during installation (Figure 2.3-9b).

Collection, or trench, lysimeters intercept and collect water as it percolates through the unsaturated zone and consist of a lined basin or trough filled with a well-drained material and placed below the area to be monitored (Figure 2.3-10). They are connected to a sample collection well or manhole. As in the case of pressure-vacuum lysimeters, leachate would be identified by analysis of line collected sample for appropriate parameters.

Use of lysimeters between double liners is redundant, since use of a double liner system with leachate collection between liners is the equivalent of installing a single collection lysimeter or innumerable pressure-vacuum lysimeters under the entire containment site. Moreover, pressure-vacuum lysimeters collect a sample from a very small region 0.03 to 0.06 $m^3$ (typically 1 to 2 $ft^3$) making it difficult and expensive to pinpoint the location of a leak. For this reason, the utility of lysimeters as a leak detection alternative is limited.

### 2.3.1.6.2 Seismic Measurements

Seismic measurements may be considered as a possible alternative for leak detection, although such systems would be costly and of questionable reliability. Seismic measurements are generally made to determine the thickness and depth of geologic layers and the velocity of seismic waves within the layers. Seismic measurements are commonly done at the surface, although they can also be made in boreholes (surface to borehole or borehole to borehole). None of these variances can be implemented in a thin drainage layer without extreme difficulty.

A feasible alternative consists of installing a network of seismometers within a sand drainage layer below a synthetic liner to measure the seismic velocity in the sand layer. This velocity is expected to increase significantly when that layer, or a portion thereof, becomes fully saturated with leachate. But factors such as implementability (up to thousands of seismometers per acre), generation of appropriate seismic signals to detect a saturated condition in a relatively thin medium (no more than several feet thick) lead to the belief that such a system would be prohibitively expensive and of questionable reliability.

### 2.3.1.6.3 Electromagnetic Techniques

Electromagnetic techniques include mutual inductance, very low frequency and high frequency techniques. Each of these has limited or no application to the problem of leak detection and is discussed briefly below.

- Mutual Inductance

This technique, also referred to as terrain conductivity measurement, measures the apparent conductivity of a volume of earth material between the transmitter and receiver coils. The depth of penetration is a function of the orientation of the coils, the separation between them, and the frequency of the electromagnetic field generated by the transmitter. Measurements are made at the surface and have been used traditionally for mineral exploration and, more, recently to trace contaminate migration at waste disposal sites. The technique is considered to be semi-quantitative and is probably not capable of detecting infiltration of leachate or impounded liquids into a thin drainage layer.

- Very Low Frequency

The very low frequency (VLF) technique has traditionally been used for mineral prospecting and utilizes the 10-20 KHz electromagnetic waves generated by the U.S. Navy for global submarine communications. The VLF technique transmitters measure the electrical properties (inductive conductivity) of the earth materials in the vicinity of the instrument. The depth of penetration can be varied by utilizing transmitters with different frequencies. VLF shows promise for plume-tracing [Waller and Davis, 1982], but its capability for detecting leachate infiltration into a thin drainage layer remains questionable.

- High Frequency

These techniques may be subdivided into pulsed and continuous wave systems and operate by transmitting energy between frequencies of 1 MHz and 1000 MHz into the materials being tested.

The pulsed wave system is more commonly referred to as ground probing radar (GPR), and is usually operated from the surface. Discrete pulses of energy are transmitted into the material to be tested. The electromagnetic waves emitted penetrate the material and are reflected from boundaries across which there is a change in the dielectric properties. The reflections are detected by a receiver and are displayed on a strip chart recording. Data may also be recorded on magnetic tape for later pullback or signal enhancement.

For the continuous wave system, the signal transmitter operates continuously making it more effective in applications where the receiver can be isolated from the transmitter by the material to be tested (as in adjacent boreholes).

The two high frequency systems tend to be expensive, both in terms of the electronics required, and the skilled staff necessary for data acquisition and interpretation. Both systems suffer from signal attenuation in the presence of conductive materials, as might be present at a waste repository. Accordingly, both high frequency techniques would seem to have very limited application to the problem of leak detection.

#### 2.3.1.6.4 Moisture Blocks

The use of moisture blocks to measure changes in soil moisture content has been widespread for many years, particularly in agricultural applications. A typical block consists of two electrodes embedded in a porous block of gypsum 16 $cm^3$ (0.4 $in^3$) in size. The blocks are responsive, typically within hours, to a wide range of changes in the electrical resistance between the embedded electrodes. The soil moisture content can be determined if the block has been calibrated to the particular soil in which it is placed.

A network of moisture blocks embedded in a drainage layer beneath a FML at a waste repository is feasible. Such an installation would probably serve as a reasonably reliable leak detection system. A recent leak detection experiment using moisture blocks embedded in a compacted subgrade demonstrated the efficacy of moisture blocks in this application [Myers et al, 1983].

Moisture blocks have drawbacks. They are fragile and require careful installation to avoid damage. Moreover, a moist environment may eventually lead to dissolution of the gypsum and system failure. A 2- to 8- year lifetime is often quoted by manufacturers for agricultural applications. Therefore, the reliability of the system may become questionable with time.

### 2.3.2 Selection of Leachate Collection and Removal System as Leak Detection System

Several technologies for leak detection were reviewed and evaluated in Section 2.3.1. These are: (1) drainage layer technology which utilizes leachate collection and removal systems (LCRS) between the liners as leak detection, collection and removal systems (LDCRS);

and (2) innovative technologies which involve electrical resistivity (ER), time domain reflectometry (TDR), acoustic emission monitoring (AEM) and other techniques. This review and evaluation showed that, for general usage, drainage layer technology currently has significant advantages over other innovative (and sometimes promising) technologies. Due to these general advantages, drainage layer technology is the method advocated to satisfy the statutory leak detection requirements of RCRA. A summary of leak detection technologies is provided below.

### 2.3.2.1 Drainage Layer Technology

LDCRS have several distinct advantages, for general application at all land disposal units, over the other methods evaluated.

- Properly designed LDCRS can provide rapid leak detection;
- LDCRS provide information on the volume of leakage. In addition, if leakage is analyzed, information on chemical constituents can be obtained. Measuring the quantity of liquids in the LDCRS requires generally unsophisticated equipment and can be done on a daily basis with relatively little effort. The leakage rate can be estimated by simple liquid level measurements at the LDCRS sump.
- LDCRS is a proven technology that has been demonstrated through actual field use.
- LDCRS are very durable. Their primary component (i.e., the drainage media) are comprised of relatively inert materials, such as clean silica sands or high density polyethylene (HDPE) synthetic drainage materials.
- LDCRS are highly reliable, low maintenance systems that can be expected to function well over time.
- LDCRS provide 100% leak detection coverage under all portions of the top liner potentially exposed to waste. The drainage system of the LDCRS is capable of detecting leakage in all areas of the bottom and sidewalls of the unit.

- LDCRS combine two important functions, leak detection and leakage collection and removal, since drainage layer technology is currently the basis for existing LCRS.

- LDCRS satisfy the requirement of a leachate collection and removal system between the liners as described in the Proposed Double Liner Rule (51 FR 10707-12, March 28, 1986).

- The use of a leachate collection and removal system between the top and bottom liners is state of practice in currently constructed waste management units. An owner or operator can continue to utilize his current design approach to meet the requirements of an approved leak detection system rather than developing new and potentially incompatible design concept for the various components. Using leachate collection and removal systems between liners to detect leaks also minimizes additional operational and cost requirements associated with leak detection.

### 2.3.2.2 Innovative Technologies

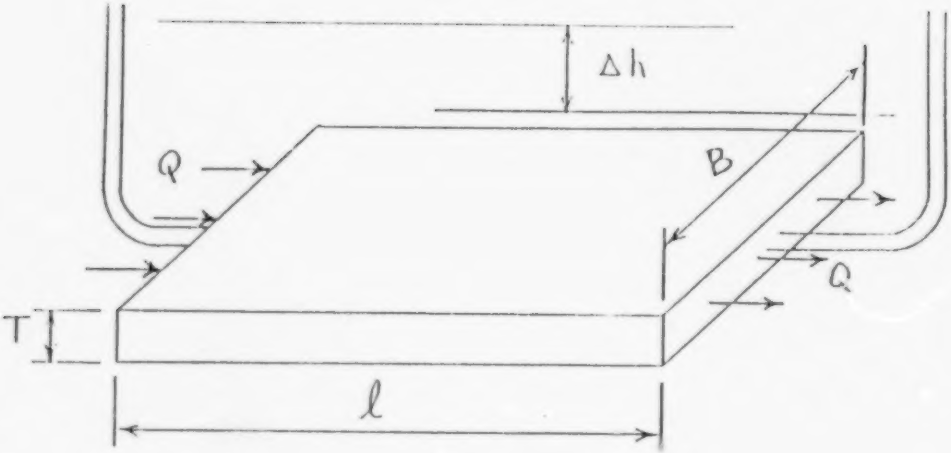
Under ideal conditions. each of the innovative technologies evaluated is capable of performing better than LDCRS with regard to leak location and detection time. However, none of these technologies directly measures leak volume nor enables leakage collection for chemical analysis. In addition, all of these technologies are still in the field testing stage and none is routinely used in permanent applications at waste management units. The long-term reliability and durability of all of the innovative systems must be further established before they can be considered for use in permanent applications.

Electrical resistivity (ER) is a geophysical technique whereby an electrical current is introduced into the ground by a pair of surface electrodes and the resultant potential field, as measured by a second pair of electrodes, is interpreted to detect anomalies (leaks). ER is limited in that it does not provide information on the rate of leakage and cannot be practically used with either composite top or bottom liners. Routine monitoring using ER techniques would be time consuming and expensive and the reliability and durability of an ER system is not well documented. ER shows promise for detecting the leak location at surface impoundments known to be leaking. It also has the potential to be used for CQA verification of certain portions of the liner such as the sump.

Time domain reflectometry (TDR) measures the electrical property variations in the material along a pair of parallel transmission line conductors. TDR is sensitive to soil moisture content making it attractive for leak detection. However, TDR has several drawbacks: (1) it must be installed in a granular drainage media with a relatively low moisture content to ensure detection sensitivity; (2) it is advantageous to use a drainage layer of well-compacted medium to fine sand because it increases horizontal dispersion of the wetted front of a leak increasing the TDR response; however, too many fines rapidly attenuate the TDR signal and also decreases the leachate collection and removal efficiency of the LCRS; and (3) the durability of TDR systems are limited due to the potential for corrosion of the exposed transmission line conductors.

Acoustic emission monitoring (AEM) detects vibrations produced by liquids leaking from a containment site through the use of transducers. The technology has not been proven at a full scale site and has several drawbacks: (1) the potential exists for sensors and wires to corrode during the active life and post-closure care period; (2) AEM may not detect small leaks or low velocity leaks where the flow is not turbulent; and (3) AEM is sensitive to background noises (i.e., nearby equipment or machinery). Also, the technology is only reliable if it identifies the leak within minutes of its occurrence. For this reason, the method must be used with a costly automated continuous monitoring system. A brief malfunction of the monitoring system would be unacceptable since any leaks starting during the downtime may not be detected by the system.

Other technologies were found to be inappropriate as a primary leak detection system for waste management units. These include lysimeters, seismic measurements, electromagnetics and moisture blocks.



Darcy's Equation for Saturated Flow Conditions:

$$Q = kiA$$

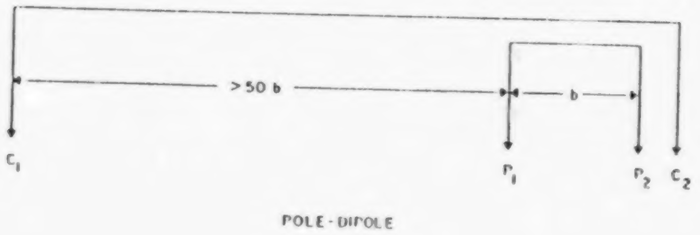
$Q$ = rate of flow ($m^3/s$ or gpm);
$k$ = in-plane hydraulic conductivity (m/s or cm/s);
$i = \Delta h/l$ = hydraulic gradient (dimensionless);
$A = TB$ = cross-sectional area in the direction of flow ($m^2$ or $ft^2$)

Darcy's Equation may be rewritten as:

$$Q = \theta i B$$

$\theta = kT$ = hydraulic transmissivity ($m^2/s$ or $ft^2/s$);
$i$ = hydraulic gradient;
$B$ = width of the cross-sectional area in the direction of flow (m or ft).

Figure 2.3-1. Definition of terms for flow in a LDCRS drainage layer.



C - CURRENT ELECTRODE
P - POTENTIAL ELECTRODE

Figure 2.3-2. Electrode configuration used to detect leaks from waste management units lined with FMLs.

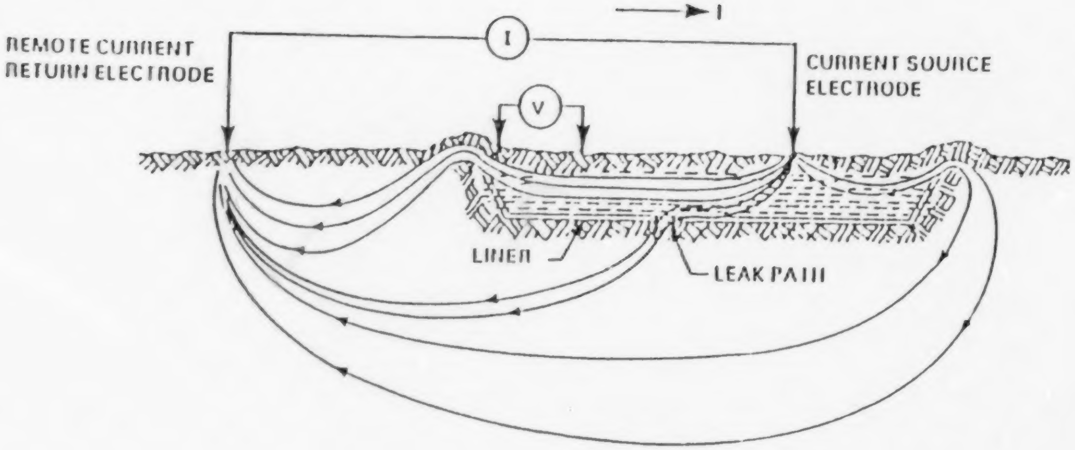


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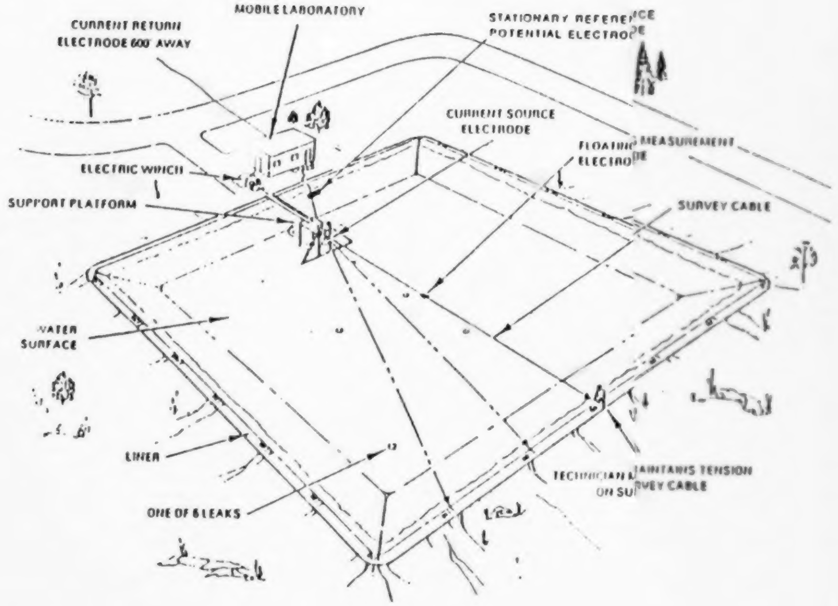
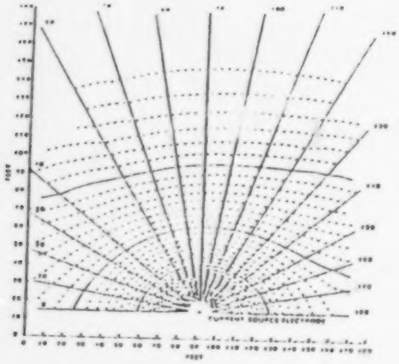
Figure 2.3-3. Conceptual electrical resistivity testing technique applied to detect and locate leaks through a FML liner. [Schultz et al., 1984]

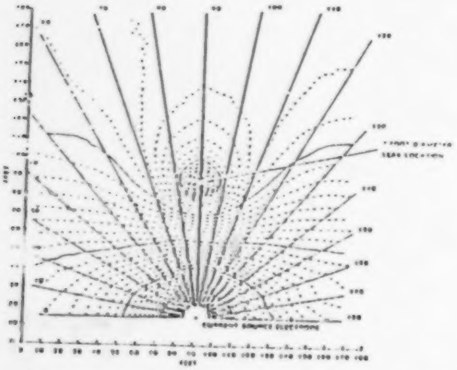

MOBILE LABORATORY
CURRENT RETURN ELECTRODE 600' AWAY
STATIONARY REFERENCE POTENTIAL ELECTRODE
CURRENT SOURCE ELECTRODE
FLOATING MEASUREMENT ELECTRODE
ELECTRIC WINCH
SUPPORT PLATFORM
SURVEY CABLE
WATER SURFACE
LINER
TECHNICIAN MAINTAINS TENSION ON SURVEY CABLE
ONE OF 6 LEAKS

NOT TO SCALE

Figure 2.3-4. Conceptual drawing of the test impoundment and operation of the test equipment used to evaluate electrical resistivity leak detection system. [Schultz et al., 1984]



An equipotential contour plot of no leak measurements



An equipotential contour plot showing the distortions from a 1 foot diameter leak on the 90° radial in approximately 9 feet of water

Figure 2.3-5. Equipotential contour plots obtained from electrical resistivity survey. [Schultz et al., 1984]

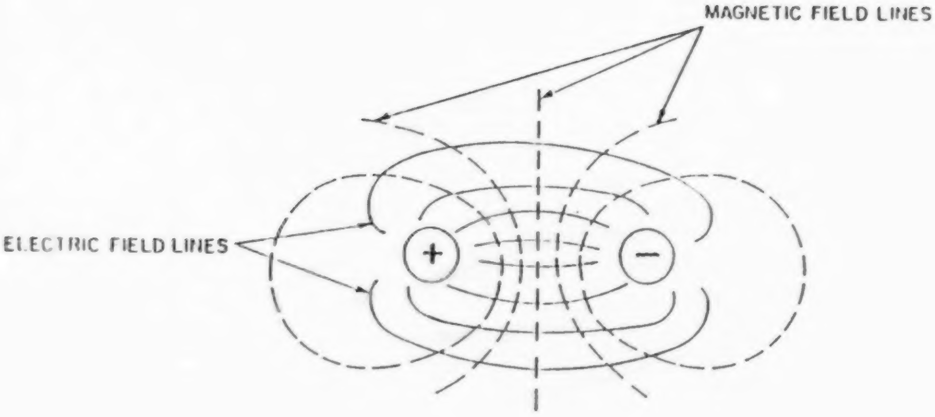


Figure 2.3-6. Parallel transmission line conductors used for Time Domain Reflectometry (TDR).

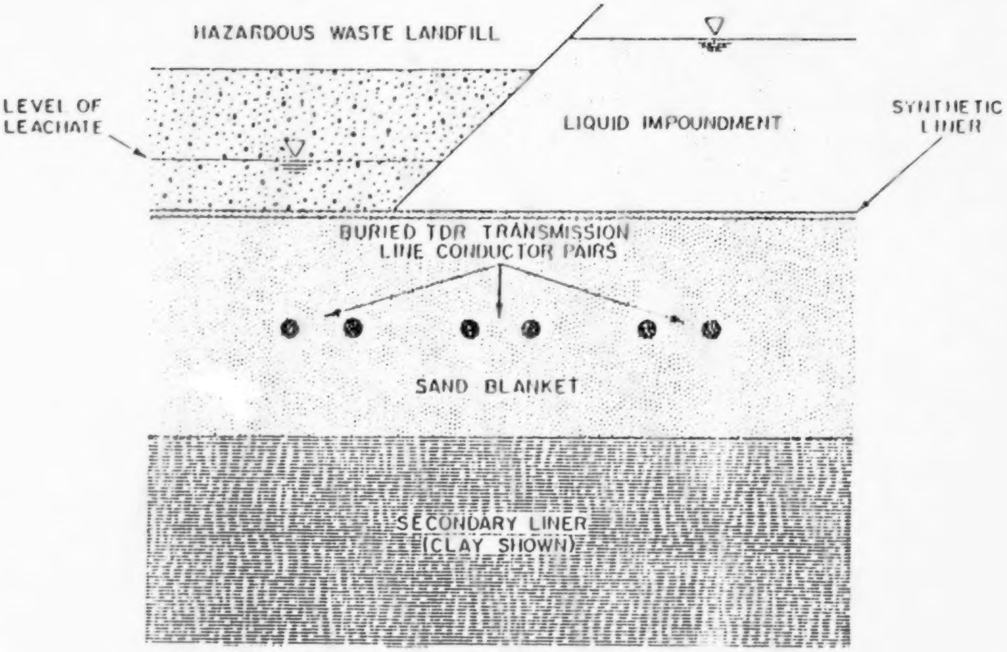


Figure 2.3-7 Conceptional TDR installation at a hazardous waste management unit.

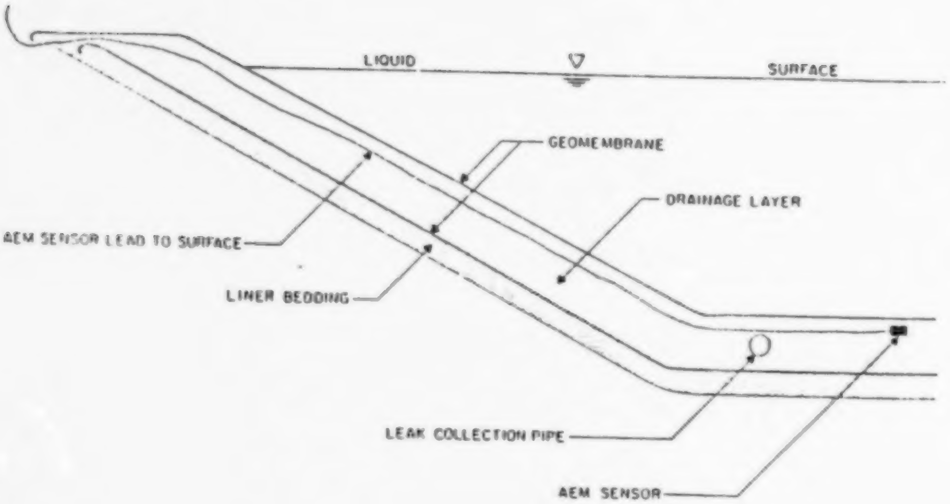


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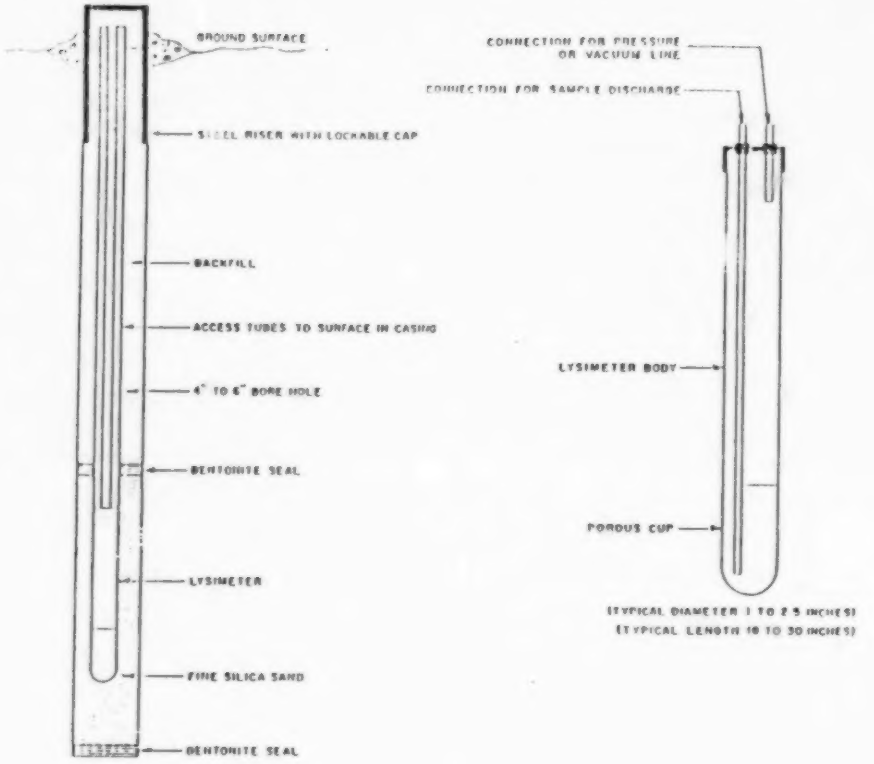
Figure 2.3-8 AEM sensor installed below the top liner at a new surface impoundment.


GROUND SURFACE
STEEL RISER WITH LOCKABLE CAP
BACKFILL
ACCESS TUBES TO SURFACE IN CASING
4" TO 6" BORE HOLE
BENTONITE SEAL
LYSIMETER
FINE SILICA SAND
BENTONITE SEAL
CONNECTION FOR PRESSURE OR VACUUM LINE
CONNECTION FOR SAMPLE DISCHARGE
LYSIMETER BODY
POROUS CUP
(TYPICAL DIAMETER 1 TO 2.5 INCHES)
(TYPICAL LENGTH 18 TO 30 INCHES)

b - Typical Installation

a - Typical Pressure-Vacuum
Lysimeter

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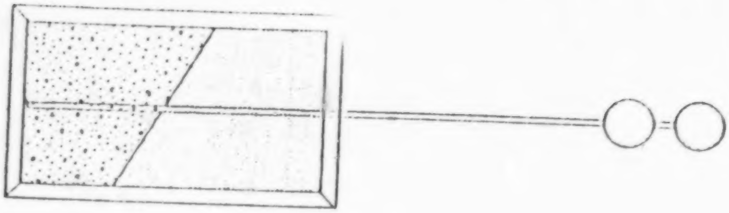
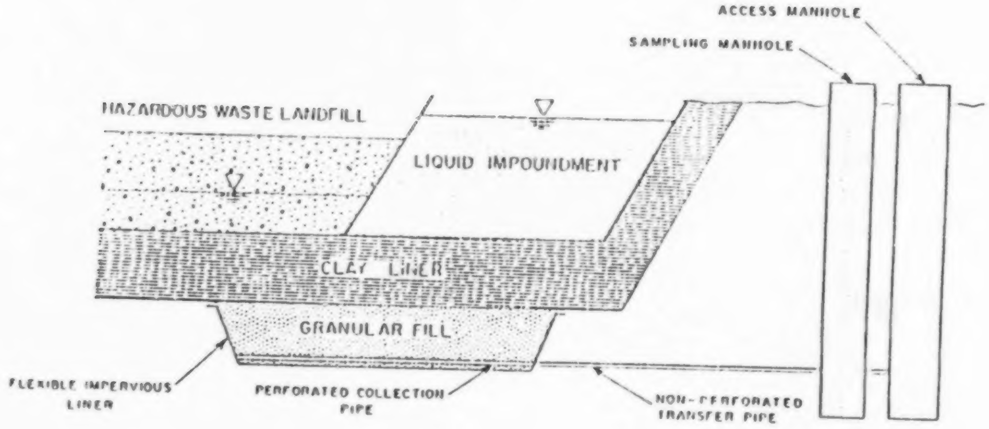
Figure 2.3-9. Schematic of a pressure-vacuum lysimeter and typical installation. [GCA, 1984b]



PLAN VIEW



SECTION

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Figure 2.3-10. Layout of a collection lysimeter. [GCA, 1986b]

## 2.4 LEAK DETECTION SYSTEMS BETWEEN LINERS

### 2.4.1 Functions of a Leak Detection System

The functions of a leak detection system between the top and bottom liner are to detect leakage through the top liner at the "earliest practicable time" and to provide information about the rate of leakage. This information will enable the owner or operator and EPA to determine what actions (such as accelerated pumping of the LDCRS sump, etc.) are required to ensure that the hydraulic head on the bottom liner is minimized (and thereby ensure that leakage into and through the bottom liner is minimized).

For a given rate of top liner leakage, the hydraulic head on the bottom liner can be minimized by removing the leachate in the LDCRS as fast as possible. Thus, the greater the leachate collection and removal capability of the LDCRS, the smaller the possibility of generating significant hydraulic heads on the bottom liner.

The key parameters to be considered in evaluating liquid collection and removal capabilities include the hydraulic conductivity and hydraulic transmissivity of the leak detection system drainage media, the hydraulic gradient (which is a function of the slope of the drainage layer), and the hydraulic conductivity of the bottom liner. As was shown in EPA's recent Notice of Availability of Data on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments" [USEPA, 1987], a compacted soil bottom liner allows significantly greater leakage into and through the bottom liner than does a composite. On this basis, a composite bottom liner is preferable to a compacted soil bottom liner. In the remainder of this document, a composite bottom liner is assumed.

The hydraulic head above the bottom liner should be kept at a minimum at all times. The hydraulic head affects the rate of leakage through the bottom liner. The larger the head, the larger the leakage rate. In the case of free surface flow, and neglecting dynamic effects, the head of liquids on the underlying bottom liner is [Giroud and Bonaparte, 1984]:

$$h = (Q/B) / (k_d \tan \beta) \qquad \text{(Equation 2.4-1)}$$

where: Q/B = rate of flow to be transmitted by the drainage layer per unit width; $k_d$ = in-plane hydraulic conductivity of the drainage material; and $\beta$ = slope of the drainage layer. Recommended SI units are: Q/B ($m^2/s$) and $k_d$ (m/s). Equation 2.4-1 indicates that for a given flow rate and drainage layer slope, the head decreases with increasing drainage material hydraulic conductivity. In the case of leak detection systems, the rate of flow to be transmitted by the drainage layer is the rate of leakage through the top liner assuming there is no other source of liquids entering the leak detection system (e.g, ground water, construction water, etc.). In-plane hydraulic conductivity is a property of the drainage media and the liquid and is typically evaluated under saturated flow conditions. In-plane hydraulic conductivity of various drainage materials is discussed in Section 2.4.3.2.

Leak detection time is the time between when leakage enters the LDCRS until the time it is collected in the LDCRS sump and available for removal. The time between the occurrence of a leak and its detection should be as small as possible. Detection time is essentially a function of travel distance and velocity. Travel distance is established by the geometrical characteristics of the land disposal unit. For a given unit, detection time decreases with increasing velocity. Key parameters affecting velocity include in-plane hydraulic conductivity of the drainage material and hydraulic gradient. Velocities are higher in materials which are highly permeable in their plane.

A leak detection system is an integral part of EPA's "liquids management strategy" and "systems approach" to waste containment. EPA's liquids management strategy has two main objectives: (i) minimize leachate generation in the waste management unit; and (ii) maximize leachate removal from the waste management unit at the earliest practicable time. It is through these two operational objectives that EPA will achieve the Congressional goal of preventing migration out of the unit.

The "systems approach" to waste containment applies specifically to the second part of the "liquids management strategy", namely, maximizing leachate removal from the land disposal unit. The double liner system is the mechanism by which leachate collection and removal can be maximized. The top and bottom liner together with the LCRS above the top liner (in the case of landfills) and the LDCRS between the liners function in an integrated, interdependent manner to prevent leachate migration out of the unit by maximizing its collection and removal. Each of the system elements reinforces and supports the other: the liners serve as a barrier to leachate migration and facilitate its collection and removal; the leachate collection and removal system (LCRS) above the top liner in landfills enables collection and removal of leachate and minimizes the buildup of the liquid pressure on the top liner; the leachate collection and removal system between the liners serves to minimize the buildup of head on the bottom liner; and the leak detection system provides the owner or operator and EPA with notification of leakage through the top liner, which enables the review of existing conditions and may lead to the taking of certain response activities.

Clearly, the LDCRS, which combines leak detection and leachate collection and removal, provides an important element to EPA's liquids management strategy.

### 2.4.2 Materials

#### 2.4.2.1 Introduction

Bonaparte et al. [1985] have identified a discussed two categories of leachate collection and removal systems: (1) those involving the use of coarse-grained granular materials exclusively or a combination of granular and synthetic materials (conventional leachate collection and removal systems); and (2) those involving the use of synthetic materials exclusively (herein called "synthetic" leachate collection and removal systems). Their discussion could be adapted to leak detection systems. Conventional leak detection systems typically consist of network of collection pipes, drainage layers made of granular materials, and collection/monitoring structures. Filters, when needed, are placed adjacent to granular drainage media in order to prevent or reduce clogging of the granular material due to intrusion of small particles from adjacent soil. Filters are especially needed when the soil adjacent to the granular drainage media consists of fine-grained soil such as the compacted soil component of a composite top liner, or a bottom liner made of compacted soil alone. These filters typically consist of fine-grained granular soils or geotextiles.

Synthetic leak detection systems differ from conventional systems in that they typically consist of a synthetic drainage layer, in lieu of granular materials, to transmit flow to a collection/monitoring structure. In addition, almost all synthetic systems must use geotextiles as filters, where necessary, to prevent adjacent soil from clogging the synthetic drainage layer.

Figure 2.4-1 illustrates the use of synthetic drainage materials to create leak detection systems.

The following sections discuss materials for the components of leak detection systems. Components include drainage layers, collection pipes, collection/monitoring structures, filter layers (where necessary) and cushion layers (where necessary) to protect FMLs.

### 2.4.2.2 Drainage Layers

#### 2.4.2.2.1 Granular Drainage Materials

A wide range of sands and gravels can be used in leak detection systems which may vary from medium to coarse in size and well-graded to uniform in gradation. Selection of the drainage materials depends on the following considerations:

- The drainage layer should be able to collect and remove rapidly liquids entering the system as a result of leakage through the top liner, as discussed in Section 2.4.1. The drainage layer should have adequate hydraulic conductivity and hydraulic transmissivity.

- The drainage material should not damage FMLs when the FMLs are in direct contact with the drainage material. This scenario may occur when sharp gravel is placed adjacent to a top liner consisting of a FML alone and/or a bottom liner consisting of a composite liner. Protection of the FML against damage may be accomplished by placing cushion layers between drainage material and FML. Cushion layers may consist of either sand or geotextile and are further described in Section 2.4.2.4. Criteria for the FML protection have been provided in USEPA [1985].

- The drainage layer should be physically compatible with filters or cushions made of fine-grained granular soils to minimize the migration of the filter or cushion materials into the drainage layer.

### 2.4.2.2.2 Synthetic Drainage Materials

Synthetic drainage layers are made of planar materials thick enough to convey fluids in their plane. Synthetic drainage materials are usually made from polymers. Typical polymers include polypropylene, polyester, and polyethylene. Formulations of these polymers can be manufactured to be highly inert to biological and chemical degradation.

Four types of available synthetic drainage materials have been identified by Giroud and Bonaparte [1984]. They are:

- Nets, which are the most widely used synthetic drainage materials in leak detection systems, with thicknesses ranging from approximately 4 mm to 7 mm (160 to 280 mils).

- Needlepunched nonwoven geotextiles, thickness 2 to 5 mm (80 to 200 mils).

- Mats, thickness 10 to 20 mm (400 to 800 mils).

- Corrugated, waffled or alveolate plates, thickness 10 to 20 mm (400 to 800 mils).

Nets (Figure 2.4-2) consist of two sets of parallel extruded polymer strands intersecting at a constant angle (generally between 60° and 90°). Strands of one set lie on top of strands of the other set, and the two sets are melt-bonded at the intersection. The two sets of strands create two sets of channels which can convey liquids. This ability to transmit flow depends on the net geometry and properties as discussed in Section 2.4.3.3.

A large variety of nets are available. They differ by: size and shape of cross section of strands; depth of channels; opening size; and nature of the polymer. Nets used as drainage materials in leak detection systems are usually made of medium density polyethylene (MDPE) or high density polyethylene (HDPE) and their geometry makes them suitable for drainage. Their strands are typically 1 to 3mm (1/24 to 1/8 in.) in height and width, and their overall thickness is almost twice the strand height (channel depth is approximately equal to strand height). Opening size is typically from 5 to 10 mm (1/4 to 1/2 in.).

Not all nets are suitable for drainage. Nets with both sets of strands in the same plane do not have channels deep enough to allow significant flow of liquid. Nets with large openings (typically larger than 10 mm (0.5 in.)) and deep channels could be used as a drainage medium between two rigid materials, such as concrete; but flexible materials, such as geotextiles and FMLs, can penetrate the channels when subjected to soil or liquid pressure and decrease the rate of liquid flow in the net.

Except in a few locations where they are in contact with coarse granular materials such as gravel, nets used for drainage should never be put in direct contact with soil. Fine soil particles would rapidly penetrate net openings and hamper flow of liquids or gases. In most cases, nets used for drainage are in contact with geotextiles or FMLs. Nets, geotextiles and FMLs can be supplied independently and installed successively. Composites comprising nets, geotextiles and/or FMLs are also available or can be custom fabricated.

Needlepunched nonwoven geotextiles (Figure 2.4-2) are formed from staple fibers or filaments extruded after melting a polymer. These fibers or filaments are arranged into a planar structure in an oriented or random pattern and are mechanically bonded by needlepunching. Needlepunching is accomplished by thousands of small barbed needles set into a board, which punch through the loose fiber web and withdraw, leaving fibers entangled.

Mats (Figure 2.4-2) are open structures which are made of coarse and rather rigid filaments with a tortuous shape, bonded at their intersections.

Corrugated, waffled or alveolate plates (Figure 2.4-2) are made by forming a plastic sheet into the desired profile. Forming techniques include extrusion, molding, pressing, etc.

### 2.4.2.3 Filter Layers

Two types of filters are typically used in engineering practice. These are granular filters and geotextile filters.

Granular filters were first introduced in the 1920's [Terzaghi and Peck, 1967] and have since been extensively used in hydraulic structures such as dams and reservoirs. They consist of a granular layer or combination of granular layers having a coarser gradation in the direction of seepage than the soil to be protected. Granular filters typically utilize materials which are more permeable than fine sands.

Geotextiles have been traditionally manufactured in two varieties, i.e. wovens and nonwovens. However, a variety of other related products (geotextile-related products) has been introduced recently on the market. A detailed discussion of available geotextiles and geotextile-related products is beyond the scope of this section and can be found elsewhere [Giroud, 1984a; Koerner, 1986].

The varieties of geotextiles which are most commonly used as filters are wovens and nonwovens. The following definitions are reproduced from [Giroud and Carroll, 1983].

Woven geotextiles are composed of two sets of parallel yarns systematically interlaced to form a planar structure. The manner in which the two sets of yarns are interlaced determines the weave pattern. The two sets of yarns are generally perpendicular, but some wovens weave at a skew angle.

Nonwoven geotextiles are formed from staple (short) fibers or filaments that are arranged, using oriented or random patterns, into a planar structure. The fibers or filaments are bonded together by one of the following process:

- Chemical bonding: a cementing medium such as glue, rubber, latex, cellulose derivative, or, more frequently, synthetic resin, is added to fix the fibers or filaments together. One thus obtains chemically bonded non-woven geotextiles.

- Thermal bonding: heat causes partial melting of the fibers or filaments and makes them adhere together at their cross-over points. One thus obtains heatbonded nonwoven geotextiles that are relatively thin, typically 0.5 to 1 mm (20 to 40 mils).

- Mechanical bonding by needlepunching: thousands of small barbed needles, set into a board, punch through the loose fiber web and withdraw, leaving fibers or filaments entangled. One thus obtains needlepunched nonwoven geotextiles that are relatively thick, typically 1 to 5 mm (40 to 200 mils) or more.

### 2.4.2.4 Cushion Layers

Cushion layers are used in leachate collection and removal systems to protect FMLs from potential damage caused by drainage materials. Potential for damage to the FML depends on a number of factors including nature of drainage material (e.g. particle size and shape for granular materials, flexibility or brittleness for synthetic materials) and compressive stresses.

Cushion layers typically consist of a few inch thick sand layer or geotextiles placed between the drainage layer and the FML. Sand cushions are exclusively used with granular drainage materials and are never placed in direct contact with some types of synthetic drainage material as this could clog the synthetic drainage layer. The particle size distribution of the cushion sand should be physically compatible with the granular drainage material to prevent migration of the sand into the drainage layer. If not, sand migration may occur under high seepage forces. Sand migration may result in either one or both of the following:

- decrease of the effective thickness of the cushion layer, thus increasing the potential for puncture of the FML;

- clogging of the leachate collection and removal system.

Geotextiles do not exhibit the drawbacks described above. Typical geotextile cushions consist of needlepunched nonwovens with a minimum mass per unit area of 400 g/m² (12 oz/yd²). Needlepunched nonwoven geotextiles have been briefly described in Sections 2.4.2.2.2 and 2.4.2.3.

### 2.4.2.5 Pipes

The primary use of pipes in leak detection systems is to collect the leachate in the drainage layer and convey it to collection/monitoring points. Pipes are also used in the construction of monitoring ports and system cleanouts.

Thermoplastic pipes (PVC, HDPE) are widely used in leak detection systems due to their wide range of chemical resistance and their lower potential for damaging FMLs. However, if the expected leachate contains chemicals which may be harmful to these materials, other pipe materials should be considered.

Pipe structural properties range from flexible to rigid. Flexible and semi-flexible pipes rely on bedding materials for much of their structural support. Bedding requirements are usually less crucial for rigid pipe.

Pipes may or may not be available factory-perforated. For perforated pipes, leakage enters through slots or circular openings. Slots or openings should be sufficiently large and spatially arranged to allow free flow of liquid, but not result in significant reduction in pipe strength. Alternatively, porous wall concrete pipe may be applicable for system use.

Pipes are available in various sizes. The diameter of the pipes is normally calculated based on the design flow quantity. Pipe diameters as small as 5 and 10 cm (2 and 4 in.) have been utilized in leak detection systems. However, 0.15-m (6-in.) diameter pipes have also been used and these pipes may simplify the system maintenance. EPA has recommended 0.15-m (6-in.) minimum diameter pipes for use in leachate collection and removal systems at land disposal units [USEPA, 1985].

### 2.4.2.6 Structures

Structures in leachate collection and removal systems can function as collection/monitoring ports, cleaning access points, or collection/removal structures. Selection of structure type will depend on system layout and site specific conditions. A description of various types of structures is presented below.

#### 2.4.2.6.1 Manholes

Manholes act as the ultimate collection/monitoring point where leachate will flow by gravity. They also serve as a cleaning access for system inspection and maintenance.

Manholes used in leachate collection and removal systems are often identical to those used in conventional construction projects. Manholes are normally precast concrete structures. However, structures manufactured using other types of material, such as high density polyethylene (HDPE), are also available. In cases where expected leachate and concrete are incompatible, a different type of material should be specified.

The diameter of the manhole should be large enough to permit personnel and equipment entry. Manholes with an inner diameter of 0.6 to 1.2 m (2 to 4 ft) have been used. Manholes may or may not be associated with sumps and they may be located inside or outside of the waste management unit boundaries.

#### 2.4.2.6.2 Sumps

Sumps are typically located at the lowest points of a land disposal unit. They essentially perform the same system functions as manholes. The benefit of sumps over manholes in leachate collection and removal systems is that they can be designed to operate between liners. Penetration of the top liner would therefore be unnecessary. Sumps are usually made of concrete. Similar to manholes, chemical compatibility of the sump material with the expected leachate must be evaluated. An alternative approach consists of lining the sump with the same type of FML specified for the top and/or bottom liners. This alternative approach might preclude the need for connections between the FML and sump which is a potential source of leakage.

#### 2.4.2.6.3 Auxiliary Cleanouts

Auxiliary cleanouts permit further cleaning access to collection laterals. They also may be designed to extend between liners to the surface. Auxiliary cleanouts can be made of materials used to make pipes and their diameter is usually equivalent to the diameter of the lateral collection pipe.

## 2.4.3 Properties of Materials

### 2.4.3.1 Introduction

The LDCRS is required to perform a number of functions, as identified in Section 2.4.1. These functions are to allow leak detection rapidly (at the "earliest practicable time"), to maximize collection and removal of liquids present in the leak detection system and to minimize the hydraulic head on the bottom liner. Two basic properties of the LDCRS drainage medium have been identified which govern the drainage performance of leak detection systems under steady-state, saturated conditions. These properties are hydraulic transmissivity and hydraulic conductivity. Additionally, each component of the LDCRS must be durable to perform its respective function during the waste management unit's active life and post-closure care period. Further, the materials used in the LDCRS should not damage the FML liners.

This section discusses properties required of LDCRS components, i.e. hydraulic transmissivity and hydraulic conductivity for the drainage layer, filter characteristics for granular or geotextile filters, durability, and mechanical effects on FMLs.

### 2.4.3.2 Hydraulic Conductivity

Hydraulic conductivity describes the velocity of liquid flow through the drainage layer under a hydraulic gradient equal to one. Since the velocity of liquid flow is directly proportional to hydraulic conductivity, hydraulic conductivity is the single most important variable controlling leak detection time. The larger the hydraulic conductivity of the drainage layer, the shorter the leak detection time.

Hydraulic conductivity of a drainage layer is the rate of flow per unit cross-sectional area of drainage layer per unit hydraulic gradient. The hydraulic conductivity of a drainage layer, $k_d$ (m/s), may be calculated as follows using Darcy's equation:

$$k_d = v/i = (Q/A)/i \qquad \text{(Equation 2.4-2)}$$

where: $v$ = apparent fluid velocity (m/s); $Q/A$ is the rate of flow per unit area of drainage layer in a cross-section normal to the direction of flow (m/s); and $i$ is the hydraulic gradient (dimensionless).

Hydraulic conductivity is almost always calculated under saturated flow conditions. When the leak detection system material is not saturated, the phenomenon of capillarity takes place and the flow of liquid is no longer governed by the above equation. The development of capillarity is linked to soil particle size distribution, which is linked to hydraulic conductivity. This is discussed in Section 2.6.2.4.2.

The two sections which follow compare granular drainage materials and synthetic drainage materials in the context of hydraulic conductivity. Typical hydraulic conductivity values are also provided.

### 2.4.3.2.1 Granular Drainage Materials

Granular drainage materials include sands and gravels, as discussed in Section 2.4.2.2.1. Typical ranges of hydraulic conductivity of granular media are:

- $k_d = 10^{-5}$ to $10^{-2}$ m/s ($10^{-3}$ to 1 cm/s) for fine, medium, coarse sands; and

- $k_d = 10^{-2}$ to 1 m/s (1 to 100 cm/s) for gravels.

Hydraulic conductivity of a granular media depends on a variety of physical factors. For unconsolidated materials, hydraulic conductivity varies directly with particle size [Todd, 1980]; the larger the particle size, the higher the hydraulic conductivity. Accordingly, as shown by the above values, gravels exhibit the highest hydraulic conductivity value. Hydraulic conductivity values for gravels are variable. Uniform size gravels generally exhibit higher hydraulic conductivity values than well-graded gravels for given approximate sizes. Well-graded gravels exhibit a broad range of conductivity values due to variations in particle size; for example well-graded, alluvial gravel often has a large content of fines which blocks flow pathways. Hydraulic conductivity values for fine to coarse sand vary from higher values for coarse sands to lower values for fine sands.

### 2.4.3.2.2 Synthetic Drainage Materials

Synthetic drainage materials include nets, needlepunched nonwoven geotextiles, mats and corrugated, waffled or alveolate plates, as discussed in Section 2.4.2. The hydraulic conductivity of synthetic drainage materials is not measured directly. It is typically calculated by dividing hydraulic transmissivity of a synthetic drainage material (which is measured through testing, as discussed in Section 2.4.3.3) by its thickness. Both hydraulic transmissivity and thickness are affected by overburden compressive stress and boundary conditions, as discussed subsequently. As a result, hydraulic conductivity is also affected by compressive stress and boundary conditions.

#### - Nominal Hydraulic Conductivity

A nominal hydraulic conductivity can be defined for the flow of water at 20°C (68°F) a hydraulic gradient of one, and the smallest compressive stress necessary to keep the sample flat (practically, 10 kPa, i.e., 200 psf). Nominal values are determined with the synthetic drainage layer placed between two smooth steel plates.

Under these conditions, the following values have been obtained from a review of the literature and from manufacturers' data:

- nets: $k_d = 0.1$ to 0.5 m/s (10 to 50 cm/s)
- thick needlepunched nonwoven geotextiles: $k_d = 10^{-4}$ to $10^{-3}$ m/s (0.01 to 0.1 cm/s)

- mats: $k_d = 0.1$ m/s (10 cm/s)
- waffles: $k_d = 0.1$ to 1 m/s (10 to 100 cm/s)

– Influence of Compressive Stress and Boundary Conditions

As previously noted, the hydraulic conductivity of synthetic drainage materials is not determined directly but is calculated from the results of hydraulic transmissivity tests. The hydraulic transmissivity of a synthetic drainage layer is strongly affected by the applied compressive stress (since the synthetic drainage layer is compressible) and by the material in contact with the drainage layer (since this material tends to penetrate the openings of the synthetic drainage layer). Since the hydraulic transmissivity is strongly affected, so is the hydraulic conductivity. The effect of compressive stress and non-rigid boundary conditions is to reduce the hydraulic conductivity of the synthetic drainage layer below its nominal value. The reduced conductivity can be calculated from the reduced hydraulic transmissivities measured in tests with non-rigid boundary materials (such as geotextiles and FMLs) and at elevated compressive stresses (simulating waste overburden stresses). Hydraulic transmissivities under these conditions are presented in Section 2.4.3.3.

### 2.4.3.3 Hydraulic Transmissivity

Hydraulic transmissivity of a drainage layer is the rate of flow per unit width of drainage layer, under a unit hydraulic gradient. The hydraulic transmissivity of a drainage layer related to a given liquid may be calculated as follows using Darcy's equation [Williams et al., 1984]:

$$\theta_d = (Q/B)/i \qquad \text{(Equation 2.4-3)}$$

where: $Q/B$ = rate of flow per unit width of drainage layer ($m^2/s$); and $i$ = hydraulic gradient (dimensionless). Recommended SI units are: v (m/s) and Q/B ($m^2$).

The hydraulic transmissivity and in-plane hydraulic conductivity of a drainage layer are related as follows:

$$\theta_d = k_d T \qquad \text{(Equation 2.4-4)}$$

where: $k_d$ = in-plane hydraulic conductivity of the drainage layer (m/s); and T = thickness (m).

The hydraulic transmissivity of a drainage layer depends on several parameters. Not all the parameters affect the transmissivity of all types of drainage materials, e.g. parameters which affect synthetic materials may have a negligible influence on granular materials. The discussions which follow have been grouped in two sections. Both the section on granular drainage materials and the synthetic drainage materials discuss:

- how hydraulic transmissivity is obtained; and
- the influence of various parameters such as compressive stress and boundary conditions on hydraulic transmissivity.

In addition, both sections provide typical hydraulic transmissivity values.

### 2.4.3.3.1 Granular Drainage Materials

Granular drainage materials include sands and gravels, as discussed in Section 2.4.2.2.1. The hydraulic transmissivity of granular drainage materials is typically calculated by multiplying the hydraulic conductivity of the drainage material by the thickness of the drainage layer, although transmissivity can be directly measured through testing. Typical ranges of hydraulic transmissivity for a 0.3 m (1 ft) thick layer of granular materials are:

- $\theta_d = 10^{-6}$ to $10^{-3}$ $m^2/s$ (0.005 to 5 gpm/ft) for sands;
- $\theta_d = 10^{-3}$ to $10^{-1}$ $m^2/s$ (5 to 500 gpm/ft) for gravels.

- Influence of Compressive Stress

Compressive stresses generally have little to no effect on the hydraulic conductivity of granular drainage materials and the thickness of granular drainage layers. Therefore compressive stresses generally have little to no effect on the hydraulic transmissivity of drainage layers made of granular drainage materials.

- Influence of Boundary Conditions

The hydraulic transmissivity of drainage layers made of granular materials is not significantly affected by boundary conditions because:

- granular drainage layers are relatively thick (compared to synthetic drainage layers); and

- the boundary materials cannot significantly penetrate the granular drainage layer (except in case of clogging).

#### 2.4.3.3.2 Synthetic Drainage Materials

Synthetic drainage materials include nets, needlepunched nonwoven geotextiles, mats and corrugated, waffled or alveolate plates, as discussed in Section 2.4.2. The hydraulic transmissivity of synthetic drainage materials is typically measured in the laboratory. Hydraulic transmissivity testing is described in detail in Williams et al., [1984]. The hydraulic transmissivity test device is shown in Figure 2.4-3. Transmissivity tests should simulate as accurately as possible the conditions that will exist in the land disposal unit by accounting for such variables as overburden compressive stress and boundary conditions. The influence of each of these parameters is discussed below.

- Nominal Hydraulic Transmissivity

A nominal hydraulic transmissivity can be defined for the following standard conditions: (i) the fluid is water; (ii) the temperature is 20°C (68°F); (iii) the hydraulic gradient is one; and (iv) the compressive stress is the smallest necessary to keep the specimen flat (as a practical matter, 10 kPa, i.e., 200 psf). Nominal values are determined with the synthetic drainage layer placed between two smooth steel plates.

Under these conditions, the following values have been obtained from a review of the literature and from manufacturers' data:

- nets: $\theta_d = 10^{-4}$ to $10^{-3}$ $m^2/s$ (0.5 to 5 gpm/ft)

- thick needlepunched nonwoven geotextiles: $\theta_d = 10^{-6}$ to $10^{-4}$ $m^2/s$ (0.005 to 0.5 gpm/ft)

- mats: $\theta_d = 10^{-4}$ to $10^{-3}$ $m^2/s$ (0.5 to 5 gpm/ft)

- waffles: $\theta_d = 10^{-3}$ to $10^{-2}$ $m^2/s$ (5 to 50 gpm/ft)

### - Influence of Compressive Stress

Compressive stresses are caused in the field by the weight of the material overlying the synthetic drainage layer: weight of the solid waste or earth located above the considered drainage layer in the case of a landfill or waste pile unit, and pressure of the impounded liquid exerted on a liner overlying a drainage layer in the case of a surface impoundment unit.

All synthetic drainage layers are compressible and their thickness and hydraulic conductivity decrease with increasing compressive stress. As a result their transmissivity decreases. The influence of compressive stress on the transmissivity and thickness of nets, needlepunched nonwoven geotextiles, mats and waffles is shown in Figure 2.4-4.

Some nets should be used with caution at compressive stresses larger than 300 kPa (6,000 psf) because their hydraulic transmissivity decreases rapidly beyond this value of the compressive stress. Williams et al. [1984] reported that the hydraulic transmissivity of two commercial nets could be 10 to 100 times greater at 10 kPa (200 psf) than at 500 kPa (10,000 psf). The hydraulic transmissivity of other nets decreases more progressively with increasing compressive stresses. These other nets can be used for compressive stresses as high as 1000 kPa (20,000 psf), as shown by test results presented by Bonaparte et al. [1985]. It can be concluded that the performance characteristics of nets (and other synthetic drainage layers) must be determined on a product specific basis.

Waffles exhibit a relatively low compressibility due to their stiffness and their transmissivity varies only slightly up to a certain critical value of compressive stress. Above this critical value, the waffle collapses. The critical value of compressive stress is typically 200 to 300 kPa (4,000 to 6,000 psf) which corresponds to a depth of earth or solid waste of approximately 10 to 20 m (30 to 60 ft).

Mats do not collapse, but they are very compressible and lose much of their hydraulic transmissivity at a compressive stress of 50 to 150 kPa (1,000 to 3,000 psf) and the manufacturer of the mats recommends rightfully that their use be limited to depths smaller than 5 to 10 m (15 to 30 ft).

Needlepunched nonwoven geotextiles do not collapse, but they are very compressible and lose much of their hydraulic transmissivity at a compressive stress of the order of 100 kPa (2,000 psf).

- Influence of the Combination of Boundary Conditions and Compressive Stress

The transmissivity of a synthetic drainage layer depends on the type of geotextile or FML adjacent to the drainage layer. As shown by Williams et al., [1984], the effect of geotextile boundaries (compared to steel plates) on the hydraulic transmissivity of a net is particularly significant. Thick, compressible geotextiles such as typical needlepunched nonwovens penetrate into net channels, which decreases the hydraulic transmissivity of the net:

- A thick compressible nonwoven geotextile with a mass per unit area of 400 g/m² (12 oz/yd²) on both sides of a net decreases the hydraulic transmissivity by a factor of 100 to 1000 compared to the nominal value.

- A thick compressible nonwoven geotextile with a mass per unit area of 400 g/m² (12 oz/yd²) on one side of a net and a rigid boundary on the other side decreases the transmissivity by a factor of 5 to 10 (and not only by a factor of two because, at each rib, the liquids goes from one side of the net to the other side where geotextile fibers slow down the flow).

Williams et al. [1984] observed that the effect of a FML adjacent to a net was less significant than the effect of an adjacent geotextile:

- A thin unreinforced FML (such as a 0.5 mm (20 mil) thick PVC) on both sides of a net decreases the transmissivity by a factor of 2.
- A thicker reinforced FML (such as a 0.9 mm (36 mil) thick Hypalon) or a thick, stiff FML (such as a 1 mm (40 mil) thick HDPE) on one side of a net has a negligible effect on the net transmissivity.

The above observations illustrate the influence boundary materials such as geotextiles or FMLs have on the transmissivity of a net. Figure 2.4-5 illustrates the effect of boundary conditions and compressive stress on the hydraulic transmissivity of a net by plotting the hydraulic transmissivity values of the following systems:

- 1.5 mm (60 mil) thick HDPE FML/ 5 mm (200 mil) thick polyethylene net / 2 mm (80 mil) thick HDPE FML; and
- 25 mm (1 in.) thick clay layer compacted to 90% standard proctor dry density / special polyester needlepunched nonwoven geotextile with a very high needling density, mass per unit area 250 g/m² (7.4 oz/yd²) / 5 mm (200 mil) thick polyethylene net / aluminum plate simulating a thick, stiff FML.

The hydraulic transmissivity values of a thick polyester needlepunched nonwoven geotextile made with continuous filaments, mass per unit area of 600 g/m² (18 oz/yd²) under compressive stress are also shown for comparative purposes. Figure 2.4-5 shows that:

- the hydraulic transmissivity of a FML/net/FML system is typically higher than the hydraulic transmissivity of the clay/geotextile/net/FML system by approximately one order of magnitude; and

- the hydraulic transmissivity of the clay/geotextile/net/FML system is typically higher than the hydraulic transmissivity of the needlepunched nonwoven geotextile by approximately two orders of magnitude.

### 2.4.3.4 Filter Characteristics

A filter is required to prevent clogging of a drainage layer when particles of the soil adjacent to the drainage layer are capable of migrating into the layer. In the case of leak detection systems, a filter will be needed when the top liner is a composite or the bottom liner consists of compacted soil alone. This filter may be either a single granular layer or a combination of granular layers having a coarser gradation in the direction of seepage than the soil to be protected, or a geotextile, as discussed in Section 2.4.2.5.

#### 2.4.3.4.1 Mechanisms of Filtration

A filter must prevent particles of the soil adjacent to the drainage layer from migrating into the drainage layer. At the same time, the filter must allow free flow of liquid from the adjacent soil into the drainage layer. Soil particles which are smaller than the pore spaces between the granular filter particles or the geotextile openings may pass through the granular or geotextile filter when they are subjected to shear stresses or a high seepage gradient. The ability of a filter to provide soil particle retention therefore depends on the size of its pore spaces or openings. The smaller the size of the pore spaces or openings, the better the retention provided by the filter. However, the size of the filter pore spaces or openings should not be too small and impede free flow of liquids. The functions of a filter may be summarized as follows: the filter must be permeable enough to allow free flow into the drainage layer (which implies that the size of pore spaces or openings must be larger than a minimum value) and at the same time retain the soil to be protected (which implies that the size of pore spaces or openings cannot exceed a maximum value).

#### 2.4.3.4.2 Granular Materials

Various design criteria are available for granular filters. The original work was done by Terzaghi [1922] and was followed by considerable work done by many researchers and organizations. As a result, various expressions of the criteria can be found. The following expression of the criteria can be found in [Cedergren, 1977].

- Retention Criterion

The 15% size ($D_{15}$) of granular filter material (i.e., the particle size larger than 15 percent of the soil particles on a dry weight basis) must not be more than four or five times the 85% size ($D_{85}$) of the adjacent soil.

$$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (soil)}} < 4 \text{ or } 5 \qquad \text{(Equation 2.4-5)}$$

Granular materials are always composed of ranges of particle sizes. The retention criterion assumes that if the pore spaces in granular filters are small enough to retain approximately the 85% of the protected soil, then the majority of the finer soil particles will also be retained (Figure 2.4-6). Exceptions to this criterion are gap-graded soils and soil-rock mixtures [Cedergren, 1977].

- Permeability Criterion

The 15% size ($D_{15}$) of a granular filter material should be at least four or five times the 15% ($D_{15}$) of a the adjacent soil.

$$\frac{D_{15}\text{ (filter)}}{D_{15}\text{ (soil)}} > 4 \text{ or } 5 \qquad \text{(Equation 2.4-6)}$$

The permeability criterion requires that granular filters be sufficiently permeable to prevent the buildup of large seepage forces and hydrostatic pressures in filters and drainage layers.

The use of granular filters implies that the drainage layer consists of natural drainage materials. In order to prevent the particles of the granular filter from migrating into the drainage layer and to ensure that free flow will occur through both the granular filter and drainage layer, both the retention and permeability criteria presented above must also be met by the drainage material-filter system. If the drainage material-filter system does not meet these criteria because the drainage material particles are too coarse, an intermediate layer may be necessary to achieve transition between the granular filter and the drainage layer.

### 2.4.3.4.3 Geotextile Filters

Geotextile filters should meet one or two requirements depending on the nature of the drainage materials:

- Granular Drainage Materials: the geotextile should meet filter criteria.

- Synthetic Drainage Materials: (1) the geotextile should meet filter criteria; and (2) under compressive stress, penetration of the geotextile into the drainage material, should be limited as penetration could reduce both the hydraulic transmissivity and conductivity of the synthetic drainage material, as discussed in Section 2.4.3.2.2.

#### - Filter Criteria

A number of filter criteria have been proposed by various authors. Most authors agree that three criteria should be considered for selecting a filter:

- A retention criterion to ensure that the geotextile openings are small enough to prevent migration of soil particles.

- A permeability criterion to ensure that the geotextile is permeable enough to allow liquids to pass almost freely through it (i.e., without significant buildup of liquid pressure upstream of the filter).

- A clogging criterion to ensure that the geotextile has a large number of openings so that blocking of a few of them will not significantly impair the performance of the filter.

These criteria are summarized in Table 2.4-1. Experience shows that filter criteria are often fulfilled by the following geotextiles:

- Nonwoven geotextiles (heatbonded or needlepunched) for fine-grained soils (i.e., soils having more than 50% of their particles smaller than 0.075 mm, i.e., US sieve No. 200).

- Woven monofilament geotextiles for sands.

Multifilament woven geotextiles and slit film woven geotextiles are often not acceptable filters, usually because they do not meet clogging criteria.

- Penetration of Net Channels

The discussion presented in Section 2.4.3.2.2 showed that the hydraulic transmissivity of some synthetic drainage materials can be significantly reduced by placing compressible or flexible geotextiles in contact with the synthetic drainage materials. Traditional needlepunched geotextiles are thick and compressible; consequently they penetrate net channels, even under low compressive stress. Unless specially required, traditional needlepunched nonwoven geotextiles should not be used in contact with some synthetic drainage materials. If a needlepunched nonwoven geotextile is necessary (for example, to meet strict filter criteria), the designer has two alternatives:

- Use two (or more) layers of synthetic drainage material with the first layer in contact with the needlepunched geotextile being neglected in the design.

- Conduct hydraulic transmissivity tests simulating the actual site conditions (as indicated in Section 2.4.3.2.2) to evaluate the transmissivity of the synthetic drainage material and needlepunched nonwoven geotextile under the simulated conditions with a needlepunched geotextile in contact.

- Specify a special needlepunched nonwoven geotextile which has been specially treated by extra needling or partial heat bonding to increase its stiffness and reduce its compressibility (these special materials are readily available from some geotextile manufacturers).

Special needlepunched nonwoven geotextiles do not penetrate net channels as much as the traditional needlepunched geotextiles discussed above. However, depending on the stiffness of the soil located adjacent to the geotextile and on the magnitude of compressive stresses, they may penetrate the synthetic drainage layer enough to significantly affect hydraulic transmissivity. Therefore, hydraulic transmissivity tests are usually recommended, even with these special materials.

Although heatbonded nonwoven geotextiles and monofilament woven geotextiles do not significantly penetrate into synthetic drainage layers, they slightly reduce the hydraulic transmissivity of the synthetic drainage layer as compared to the hydraulic transmissivity measured between steel plates. Therefore, hydraulic transmissivity tests are recommended even with heatbonded nonwovens and monofilament wovens.

### 2.4.3.5 Durability

The term durability indicates resistance to progressive deterioration. Durability of the components of LDCRS depends on the action exerted by external forces or materials (leachate), on the raw material, and on the physical structure of the material. The type of action exerted by external forces can be mechanical (abrasion, fatigue) or physico-chemical-biological (degradation).

#### 2.4.3.5.1 Abrasion and Fatigue

Granular materials generally have good resistance to abrasion and fatigue and this is usually not a concern in waste management applications.

Abrasion and fatigue resistance of a synthetic drainage material in a given application can be evaluated by tests simulating actual environmental and loading conditions. Presently there are no widely accepted standard procedures for such tests. Geotextiles with high abrasion resistance are available.

### 2.4.3.5.2 Physico-Chemical-Biological Degradation

Physico-chemical-biological degradation of LDCRS components can be caused by external agents, such as: contact with soil, atmospheric conditions (including exposure to the sun), and contact with chemicals or biological agents.

- Contact With Soil

Granular materials are naturally occurring soil materials. As a result, they are not affected by contact with soil.

Experience indicates that geotextiles (usually made of polypropylene or polyester) have a durability in excess of several dozens of years when placed in most naturally occurring soil environments. Nets, mats and waffled structures are made of polyethylene or polypropylene. Polyethylene and polypropylene both tend to be very stable polymers when placed in contact with naturally occurring soils. Since, additionally, the strands of nets, filaments of mats, and thickness of waffled structure sheets are much coarser than the fibers or filaments of a geotextile, the durability of nets, mats and waffled structures buried in the ground is expected to be equal to or greater than the durability of geotextiles buried in the ground.

- Outdoor Exposure

Granular materials do not degrade significantly with outdoor exposure.

Woven and nonwoven geotextiles are usually very sensitive to ultra violet (UV) light and should not be exposed to sunlight for a period of time exceeding a few weeks to a few months depending on the type of polymer and UV resistant additives used to manufacture the geotextile. For longer duration of exposure the geotextile should be covered with a protective material (e.g., soil).

Mats and waffled structures are usually made of polypropylene. The filaments of mats and thickness of waffled structures are much coarser than the fibers or filaments of a geotextiles. As a result, their UV resistance is equal to or greater than that of geotextiles.

Nets have coarse strands and typically contain a certain amount of carbon black which has been added to the polyethylene resin to provide UV stabilization. As a result, they can be exposed to UV light for extended periods of time.

- Contact with Chemicals

Each component of the LDCRS must be chemically resistant to attack by the expected waste liquids during the active life and post-closure care period of the unit. LDCRS components include drainage layers, filters, cushions, and collection pipes and structures. The materials involved are granular drainage materials, synthetic drainage materials, granular filters, geotextile filters, sand cushions, geotextile cushions, plastic pipes, structures and pipes made of other types of material, as discussed in Section 2.4.2.

These materials may be divided in several categories:

- Granular Materials, which include granular drainage materials, granular filters, and sand cushions.
- Synthetic drainage materials, which include nets, needlepunched nonwovens, mats and waffled structures.
- Geotextiles, which include geotextile filters and geotextile cushions.
- Plastic pipes, which include PVC, HDPE.
- Structures and pipes made of other types of materials.

This section is intended to provide the owner or operator with information on chemical compatibility of each component of the LDCRS. The chemical compatibility of system components is typically evaluated based on a two-stage process procedure: (1) knowledge of the chemical resistance of component materials which enables a preliminary selection of component material; and (2) "accelerated testing" immersion of the component in the expected waste leachate for extended periods of time and testing to evaluate property changes (this second stage enables the owner or regulatory agency to assess the performance of the component during the active life and post-closure care period with some degree of confidence). Each of the following subsections provides first a brief discussion on the chemical compatibility of the materials identified above and second references currently available [illegible]sting procedures.

Granular Materials. Granular materials are composed of a variety of minerals such as quartz and limestone. The chemical resistance of granular materials depends on their mineral content and physical structure. The chemical compatibility of granular materials with the expected waste liquids has not received much attention from designers or engineers to date, primarily based on the belief that granular materials are natural and, as such, will last forever. This belief is questionable even when one considers natural materials in a natural environment. The formation of solution cavities in limestone is a typical example that illustrates the attack of natural materials by natural agents in a natural environment. The belief becomes even more questionable when natural materials are in a chemical, man-made environment.

To meet the requirement that granular materials be chemically resistant to the expected waste liquids, Landreth [1987] suggested a testing procedure for leachate collection systems which may be adapted to leak detection systems:

> "When rock or gravel is used as drainage material in the leachate collection system, the owner/operator should verify that the mineral content of the rock is compatible with the waste/leachate mixture. The owner/operator will need to demonstrate that the rock will not be dissolved or form a precipitate that would clog the leachate collection system."

Synthetic Drainage Materials. The chemical compatibility of synthetic drainage layers is poorly documented. Needlepunched nonwoven geotextiles, mats and waffled structures are made with polypropylene or polyester. Both of these polymers have a high resistance to a wide range of chemicals. They can both be attacked, however, by chemicals from certain chemical families.

Nets are made of polyethylene. Polyethylene FMLs such as high density polyethylene (HDPE) are most often selected to line hazardous waste management units because of their resistance to attack by a wide range of chemicals. For this reason, and because the thickness of their strands is comparable to the thickness of HDPE FMLs, nets are also expected to exhibit a high resistance to attack by a wide range of chemicals.

Testing of synthetic drainage layers should be performed to evaluate their chemical compatibility. Testing should include immersion of synthetic drainage layer specimens in the expected waste liquids for extended periods of time in accordance with USEPA Method 9090. Specimens should be removed at time intervals specified by Method 9090 for hydraulic transmissivity testing. Testing should be conducted on specimens that have been immersed and on a control specimen (i.e., a specimen that has not been immersed) to evaluate the effect of immersion on synthetic drainage layer hydraulic transmissivity.

Geotextiles. As discussed previously, the polymers used to manufacture geotextiles, polypropylene and polyester, have high resistance to a wide range of chemicals. In a given category of polymer, however, woven geotextiles are usually expected to exhibit a better chemical resistance than nonwoven geotextiles because of their usually thicker yarns.

A preliminary selection of candidate geotextiles should be performed using available chemical compatibility data available from polymer manufacturers. Once a given type of geotextile has been selected, specimens of the selected geotextile should be immersed in the expected waste liquids in accordance with USEPA Method 9090. Specimens should be removed at specified time intervals for index property testing. Testing should be conducted on specimens that have been immersed, and on control specimen (i.e., a specimen that has not been immersed) to evaluate the effect of immersion on the geotextile properties. The tests to be performed should include as a minimum the

ASTM D1682 grab strength test, ASTM D751 puncture strength test, and ASTM D1004 trapezoid tear strength test.

Plastic Pipes. The most commonly used category of plastic pipes in waste management units are the thermoplastic pipes. Thermoplastic pipes include polyvinyl chloride (PVC) and high density polyethylene (HDPE). PVC and HDPE are known for their high resistance to a wide variety of chemicals. HDPE pipes are expected to exhibit a chemical resistance equivalent to HDPE FMLs. PVC pipes are expected to exhibit a chemical resistance superior to the chemical resistance of PVC geomembranes since pipes are made from almost pure PVC, while geomembranes are made from plasticized PVC, i.e., PVC mixed with plasticizers whose chemical stability is not always as good as PVC's.

The chemical resistance of other plastic pipes should be evaluated based on available compatibility data.

Testing of plastic pipes (thermoplastic or others) should be performed to evaluate their ability to withstand loads after immersion with expected waste liquids. The testing procedure which follows has been adapted from Landreth [1987]. Specimens of the plastic pipe should be prepared for strength testing per ASTM D2412 or equivalent and at least one prepared specimen should be immersed in the expected waste liquids in accordance with USEPA Method 9090. After the immersion test, the pipe specimen should be dried (per Method 9090) and subjected to a strength test (ASTM D2412, paragraphs 6-9). Testing of an identical non-immersed (control) specimen should be performed. A report should be prepared similar to that outlined in ASTM D2412 paragraph 11 (including 11.1.7 and 11.1.9) comparing the test results of the immersed and control samples.

Structures and Other Pipes. Structures and other pipes have been identified in Section 2.4.2.5 and 2.4.2.6. They are made with either one of the following materials: plastics, reinforced concrete, metals, fiberglass, and acrylonitrile-butadiene-styrene. The chemical compatibility of the structures and other pipes should be evaluated with as much care as exercised for other components of leak detection systems. Ideally, compatibility testing should be performed on every one of these structures and other pipes.

However, such testing may be impractical to perform because of the large variety of parameters involved:

- Shape of structure: some structures maybe circular (e.g. manholes), others may be quadrangular (e.g. sumps).

- Size of structure: some manholes may have a 1.2 m (4 ft) inner diameter.

- Type of material.

Available literature on the chemical behavior of these structures and other pipes should be used to evaluate their compatibility with the expected waste liquids.

### 2.4.3.6 Mechanical Effects of Drainage Materials on FML Liners

#### 2.4.3.6.1 Granular Drainage Materials

Sand and small rounded gravel should not damage FMLs. In fact, sand cushions are often used to protect FMLs, as discussed in Section 2.4.2.6. Angular gravel (e.g., crushed aggregate) can abrade, scratch, and puncture FMLs. A sand or geotextiles cushion should always be placed between angular gravel and the FML.

All granular drainage materials are placed using equipment such as bulldozers or front-end loaders. Stress generated by the wheels or the tracks and misuse of the equipment can damage the FML.

#### 2.4.3.6.2 Synthetic Drainage Materials

Needlepunched nonwoven geotextiles and mats do not damage FMLs even under relatively high compressive stresses because they are flexible and smooth. Under low compressive stresses, nets will not damage FMLs. Under high compressive stresses, nets can damage FMLs if the net is stiffer than the FML. Nets should not be used with some types of FMLs unless a cushion layer is placed between the net and the FML. Waffled structures collapse in a brittle mode at a compressive stress of 200 to 300 kPa (4,000 to 6,000 psf) and can damage FMLs. For this reason waffle structures are usually not used with FMLs in waste management applications.

Unlike granular drainage materials, all synthetic drainage materials are placed by hand, which reduces the potential for damage to the underlying FML by construction equipment. However, care should be exercised not to entrap stones nor leave other sharp objects in the synthetic drainage material during its placement.

The potential for damage common to all available synthetic drainage materials results from placement of the low-permeability soil component of a composite top liner. Synthetic drainage layers are relatively thin and do not protect the FML from construction equipment-generated stresses as a 0.3 m (1 ft) thick drainage layer made of natural materials would. A loosely compacted lift of low permeability soil should be initially placed on top of the geotextile covering the synthetic drainage layer to protect the underlying FML, until subsequent lifts of the top liner low-permeability soil can be placed and compacted.

### 2.4.4 Conclusions

This section of the background document has provided a review of the materials used to construct leachate collection and removal systems. This section has also presented a discussion of the properties of these materials, the factors that affect the materials, and typical properties. The important hydraulic properties of drainage media were listed as hydraulic conductivity and hydraulic transmissivity and ranges were given for both of these properties for both granular and synthetic drainage media. These ranges of properties are important because the limits of the range define the limits of performance of LDCRS type leak detection systems. The values of the parameters used to define BDAT performance must fall within the ranges of properties defined in this section. The ranges of hydraulic conductivities ($k_d$) and hydraulic transmissivities ($\theta_d$) reported in this section for granular and synthetic drainage materials were ($\theta_d$ for sands are based on a 0.3-m (1-ft) thickness):

- sand

  $k_d = 10^{-5}$ to $10^{-2}$ m/s ($10^{-3}$ to 1 cm/s)

  $\theta_d = 10^{-6}$ to $10^{-3}$ m$^2$/s (0.005 to 5 gpm/ft)

- gravel

  $k_d = 10^{-2}$ to 1 m/s (1 to $10^2$ cm/s)

  $\theta_d = 10^{-3}$ to $10^{-1}$ $m^2/s$ (5 to 500 gpm/ft)

- synthetic drainage materials (nominal values)

  $k_d = 10^{-4}$ to 1 m/s ($10^{-2}$ to $10^2$ cm/s)

  $\theta_d = 10^{-6}$ to $10^{-2}$ $m^2/s$ (0.005 to 50 gpm/ft).

Table 2.4-1. Filter criteria for geotextiles, adapted from [Christopher and Holtz, 1984; Giroud, 1982]. Note: $O_{95}$ is the apparent opening size of the soil.

1. RETENTION CRITERION

1.1 Soils with less than 50% particles < 0.075 mm (US Sieve 200)

| | Density index of the soil (Relative density) | Coefficient of uniformity of the soil | |
|---|---|---|---|
| | | $1 < C_u < 3$ | $C_u > 3$ |
| loose soil | $I_D < 35\%$ | $O_{95} < C_u d_{50}$ | $O_{95} < \frac{9}{C_u} d_{50}$ |
| medium dense soil | $35\% < I_D < 65\%$ | $O_{95} < 1.5\, C_u d_{50}$ | $O_{95} < \frac{13.5}{C_u} d_{50}$ |
| dense soil | $I_D > 65\%$ | $O_{95} < 2 C_u d_{50}$ | $O_{95} < \frac{18}{C_u} d_{50}$ |

1.2 Soils with more than 50% particles < 0.075 mm (US Sieve 200)

$O_{95} \leq 210$ micrometers (US Sieve 70), and

$O_{95} \leq d_{85}$ (for wovens) or $O_{95} \leq 1.8\, d_{85}$ (for nonwovens)

2. PERMEABILITY CRITERION

2.1 Critical and/or Severe Applications

$k_{geotextile} > 10\, k_{soil}$

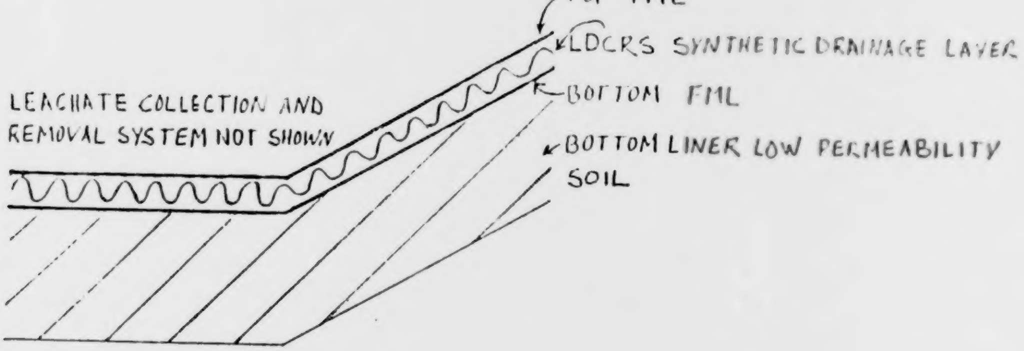
2.2 Non-critical and Non-severe Applications

$k_{geotextile} > k_{soil}$

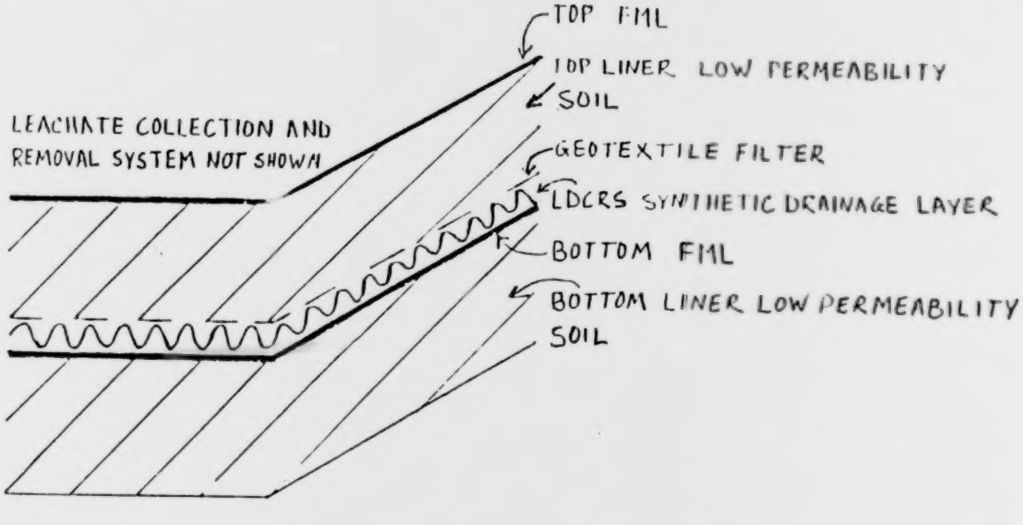
3. CLOGGING CRITERION

Nonwoven geotextiles: porosity $n > 30\%$

Woven geotextiles: percent open area $A > 4\%$

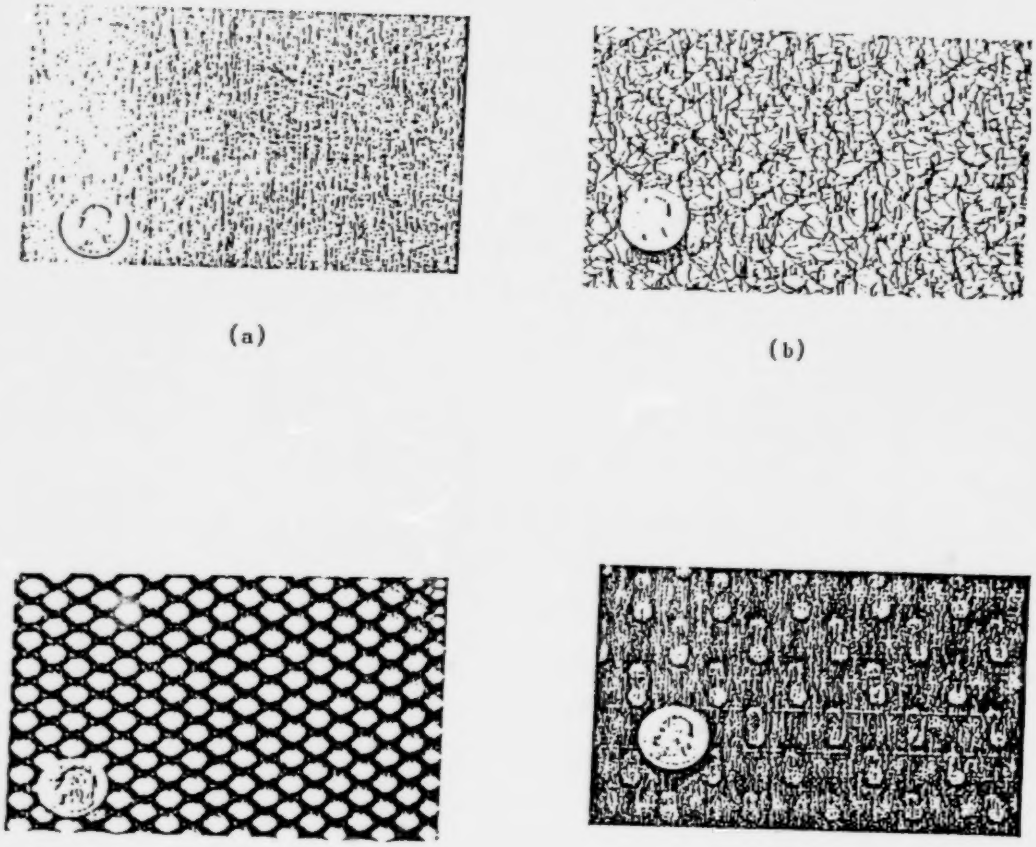

TOP FML
LDCRS SYNTHETIC DRAINAGE LAYER
BOTTOM FML
BOTTOM LINER LOW PERMEABILITY SOIL
LEACHATE COLLECTION AND REMOVAL SYSTEM NOT SHOWN

a. TOP LINER CONSISTING OF A FML ALONE


TOP FML
TOP LINER LOW PERMEABILITY SOIL
LEACHATE COLLECTION AND REMOVAL SYSTEM NOT SHOWN
GEOTEXTILE FILTER
LDCRS SYNTHETIC DRAINAGE LAYER
BOTTOM FML
BOTTOM LINER LOW PERMEABILITY SOIL

b. COMPOSITE TOP LINER

Figure 2.4-1. Synthetic leak detection collection, and removal systems.



(a)

(b)

(c)

(d)

Figure 2.4-2. Available synthetic drainage materials: (a) needlepunched nonwoven geotextile; (b) mat; (c) net; and (d) waffle. Scale: the diameter of the coin is 24 mm (1 in.) [Bonaparte et al., 1985].

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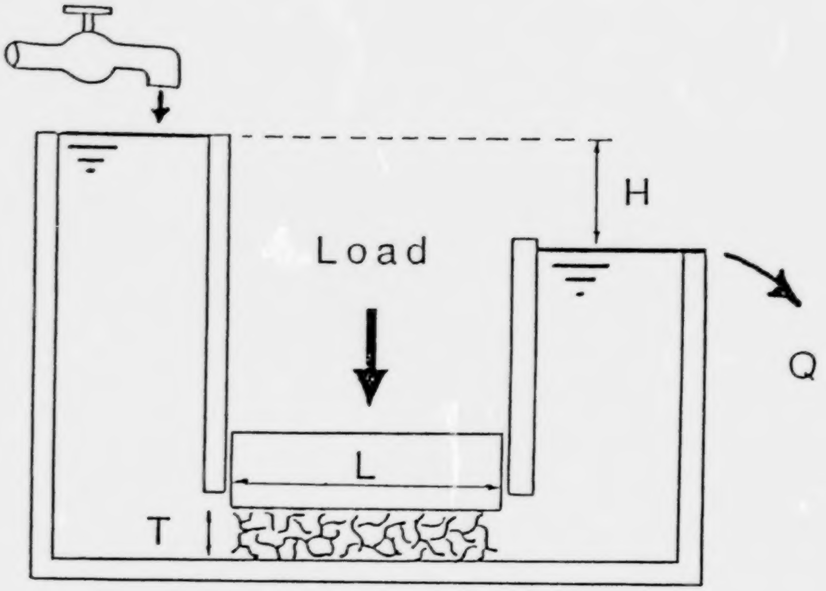

$$\theta_d = \frac{Q/B}{i} = \frac{Q/B}{H/L} = k_d T$$

Figure 2.4-3. Hydraulic transmissivity test device [Giroud and Bonaparte, 1984].

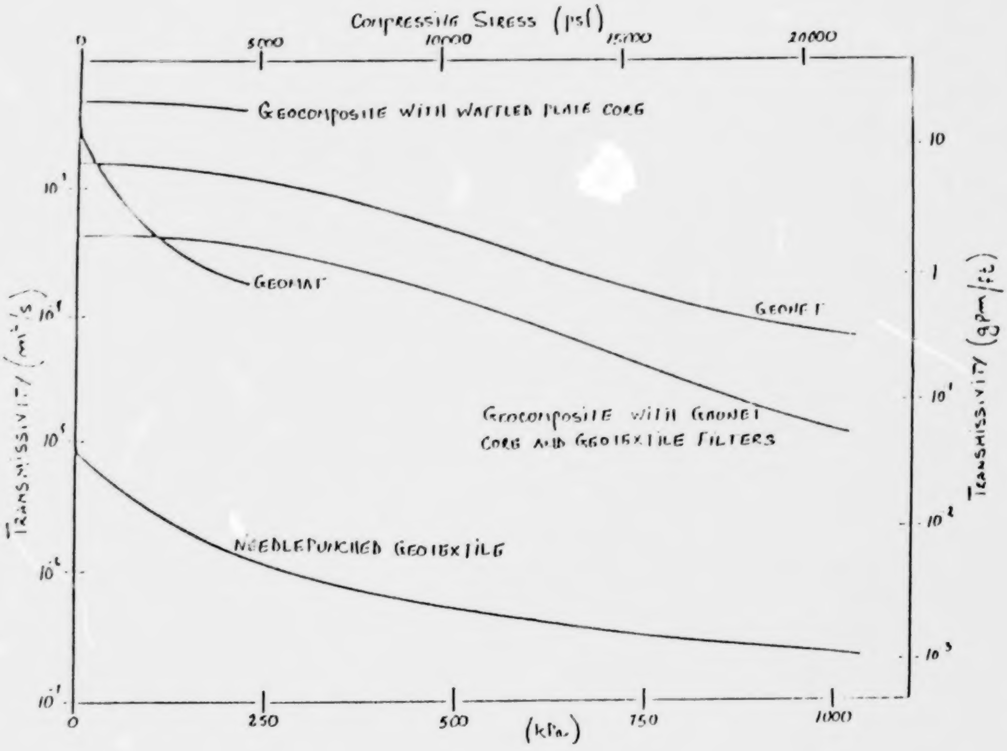

Figure 2.4-4. Influence of compressive stress on synthetic drainage material hydraulic transmissivity and thickness [Giroud, 1987].

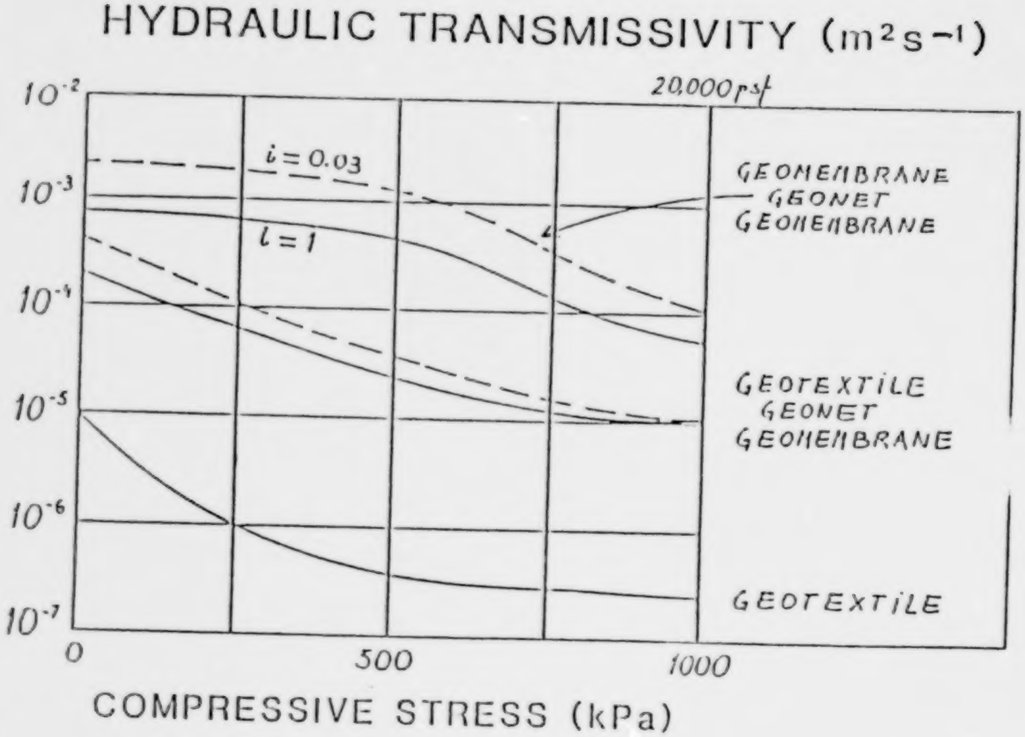

Figure 2.4-5. Influence of boundary conditions and compressive stress on hydraulic transmissivity of nets. Note: Chart established by GeoServices using values published by Bonaparte et al. [1985].

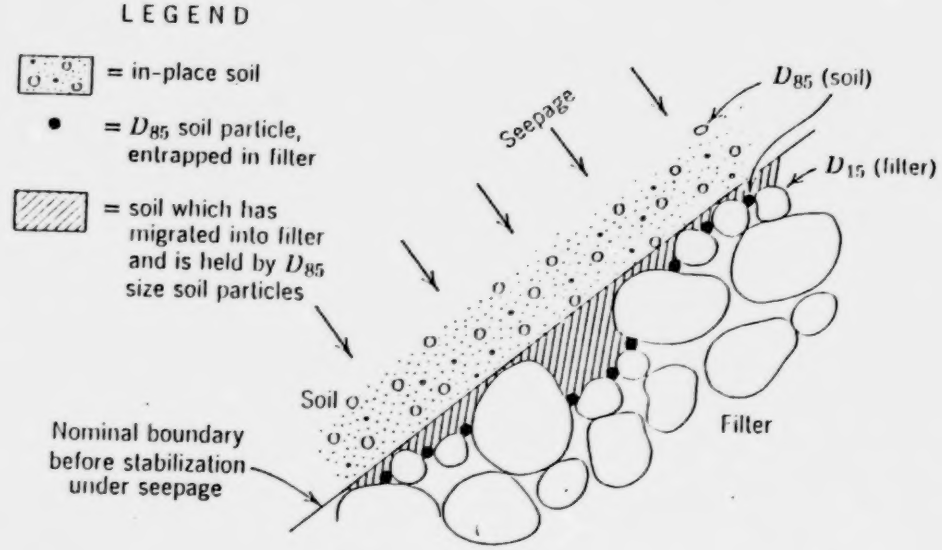
LEGEND
= in-place soil
= $D_{85}$ soil particle, entrapped in filter
= soil which has migrated into filter and is held by $D_{85}$ size soil particles
Seepage
$D_{85}$ (soil)
$D_{15}$ (filter)
Soil
Filter
Nominal boundary before stabilization under seepage

Figure 2.4-6. Illustration of prevention of piping [Cedergren, 1977].

## 2.5 LIQUIDS MEASURED IN LDCRS AT OPERATING UNITS

### 2.5.1 Introduction

To date, only very limited data have been gathered on the amount of liquids collected in the LDCRS sumps of land disposal units with double liner systems designed and constructed to current standards. Undocumented claims from a number of sources indicate that the amount of liquids collected in facilities constructed to current standards is small. However, without documentation, and without a clear understanding of the design, construction and operation of each individual unit, interpretation of data on the amount of leakage collected is extremely difficult. While the initial undocumented reports are encouraging, more (and better documented) data must be solicited from owners and operators so that the performance of double liner systems, and top liners and LDCRS in particular, can be documented and understood.

In the remainder of this section, currently available, documented information is reviewed.

### 2.5.2 Institute of Chemical Waste Management Data

In September 1986, the Institute of Chemical Waste Management (ICWM) issued a report providing data on the quantity of liquids collected in LDCRS sumps at four double-lined landfills and one double-lined surface impoundment. Data were provided on lining system design details and quantities of liquids collected in the LCRS above the top liner and LDCRS between the top and bottom liner. No information was provided on the geographic location or site hydrogeology of the five units described in the ICWM report.

The lining system details for the four landfills and the surface impoundment of the ICWM report are shown in Figure 2.5-1. It can be seen that in all cases the lining systems utilized composite top and bottom liners (a double-composite double liner system). In three of [illegible]

provides additional information on each of the units included in the

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ICWM study. An important point to be observed in Table 2.5-1 is that at four of the five facilities construction had been completed for a year or less. Table 2.5-1 provides a summary of the ICWM LDCRS data. These data indicate that the average rate of liquid collection in the LDCRS sump at the units varied from 1.5 to 18.9 liters/1000m²/day (1.5 to 18.9 gallons/acre/day), with three of the four reported values being below 6.7 liters/1000m²/day (6.7 gallons/acre/day). The maximum rate of liquid collection for each unit (based on a monitoring period of one week) is also reported in Table 2.5-1. It can be seen that, with the exception of Landfill D, the maximum rate of liquid collection tends to be on the order of several times the average rate of collection. In Landfill D, the maximum rate of liquid collection is almost an order of magnitude larger than the average rate.

Overall, the amounts of liquid collected in the units reported by ICWM are small. The following conclusions can be drawn if the leak detection systems at the considered units have been properly designed and constructed:

- The hydraulic heads acting on the bottom liners of all of these units are very small or negligible.

- Since the hydraulic heads on the bottom liners are very small, and the bottom liner at each unit is a composite, leakage into the bottom liner at each unit should be extremely small or negligible.

While it can be inferred (provided that design and construction are adequate, as indicated above) that leakage into the bottom liner at each unit is extremely small or negligible, the data presented in Table 2.5-1 allows only limited inferences to be made on the rate of leakage through the top liner into the LDCRS since:

- Landfills A, B and D contain a sand LDCRS drainage medium having a hydraulic conductivity on the order of $10^{-4}$ m/s ($10^{-2}$ cm/s). As will be shown in Section 2.6, a sand with this hydraulic conductivity exhibits significant capillarity (See Table 2.6-4). As a result leakage through the top liner will be held by capillary tension in the pore space of the sand. Drain flow will not occur until the sand has absorbed enough water to fill much of its pore space. As indicated in Section 2.6, for a sand with $k_d = 1 \times 10^{-4}$ m/s ($1 \times 10^{-2}$ cm/s), the time required to fill this pore space can be as large as several years, even for relatively long top liner leakage rates such as 100 Ltd (100 gpad). Since Landfills A, B and D are only 1 to 2 years old, leakage that has occurred through their top liners may not have yet appeared in the LDCRS sump.

- In all five ICWM units, the liquids collected in the LDCRS sump may not be only due to top liner leakage. The liquids in the sumps may be due to other sources of liquids such as rainwater, ground-water infiltration, and/or water expelled by consolidation from the compacted soil component of the composite top liners. In fact, it will be shown in Section 2.10 that liquid quantities in excess of those cited in the ICWM report can be entirely accounted for by consolidation of the compacted soil component of the top liner.

- One conclusion can be drawn for Landfill C and Surface Impoundment A. Since synthetic drainage nets are used as the LDCRS drainage medium, there is no significant capillary tension in the pore space of the leak detection layer. Since there is no capillary tension, there is also no capillary rise and thus the initial "wetting up" period for the drainage material in these units is minimal. Therefore, little liquid is being held in storage in the LDCRS and the quantity of liquids collected can be considered as an upper-bound of the actual top liner leakage. In addition, since there is a strong possibility that the collected liquids are from sources other than top liner leakage, the actual top liner leakage rate is probably significantly less than the values cited in Table 2.5-1.

It will be useful to monitor the units cited in the ICWM report over time. Continued monitoring is strongly encouraged.

### 2.5.3 Case Study - Landfill in South East U.S.

An industrial company designed and constructed a double-lined landfill for the purpose of containing solid hazardous waste. The site is located in the South East U.S. The ground-water table elevation is below the bottom of the unit. The lining system of the unit shown in Figure 2.5-2 is composed of the following components (from top to bottom):

- 0.15-m (0.5-ft) thick sand layer;
- geotextile filter;
- 0.3-m (1-ft) thick sand LCRS;
- 2-mm (80-mil) thick HDPE top FML;
- 0.3-m (1-ft) thick sand LDCRS; and
- composite bottom liner composed of (from top to bottom):
  - geotextile filter;
  - 1-m (3-ft) thick compacted clay; and
  - 1-mm (40-mil) thick HDPE FML.

(It should be noted that in this lining system cross-section, the order of the components of the bottom composite liner are reversed (compacted soil on top of FML) from the order normally used for composite liners (FML on top of compacted soil). The reason for this configuration is unknown.)

The landfill is approximately 39 m by 145 m (128 ft by 474 ft), and the LCRS and the LDCRS are continuous across the landfill. Liquids are removed from the LCRS and LDCRS at ten outlets each with each outlet draining an area of approximately 560 $m^2$ (6000 $ft^2$). In the following discussion the outlets are referred to as LCRS outlets 1 through 10 and LDCRS outlets 1 through 10.

The quality assurance of installation was performed by the owner and details regarding the installation procedure of the bottom liner, geotextile filter or HDPE top liner are not available. However, it is known that the LDCRS sand was dumped in 150 mm (6 in.) lifts, saturated and compacted with a vibratory roller. During the installation of the top liner, the LDCRS sand was allowed to drain, but, during this period, rainfall repeatedly infiltrated the LDCRS, and it is possible that the LDCRS was nearly or fully saturated at the time of top liner installation.

After installation of the top FML, but prior to installation of the LCRS sand, the landfill was flooded with water to an average depth of 0.6 m (2 ft). (The depth across the landfill varied because of the 2% bottom slope.) The landfill was flooded for a two month period and the LDCRS was monitored. Initially, leakage was detected in all ten LDCRS outlets, with the rate of leakage decreasing with time. The initial average rate of leakage observed was 0.9 liters/minute (340 gallons/day) per outlet. After approximately 1 month, leakage from LDCRS outlets 6,8,9, and 10 had stopped, leakage rates from LDCRS outlets 1,3, and 5 were decreasing to insignificant levels, and leakage rates from LDCRS outlets 2 and 4 were fairly constant at rates of 0.7 liter/minute and 0.3 liter/minute (260 gallons/day and 110 gallons/day) respectively. The rate of leakage from LDCRS outlet 7 was erratic.

The leakage that was initially collected at all ten LDCRS outlets is attributed to drainage of existing water in the LDCRS sand and compression of the sand under the surcharge of the water ponded on the top FML. The leakage from outlets 1, 3 and 5 that was observed later on was not attributed to leakage through the top FML, but to continued drainage of the LDCRS. However, the leakage observed at LDCRS outlets 2 and 4 was most likely due to leakage through holes in the top FML.

An equivalent hole size can be backcalculated from the leakage rates observed from LDCRS outlets 2 and 4, using Bernoulli's Equation for flow through an orifice (see Section 2.2). Bernoulli's Equation is applicable because there is only a small surcharge on the FML (the weight of water) and the FML will float slightly in the vicinity of the hole. The rate of leakage observed for LDCRS outlet 2 (0.7 liter/minute or 260 gallons/day) corresponds to a hole 2.7 mm (0.1 in.) in diameter. The rate of leakage observed for LDCRS outlet 4 (0.3 liter/minute or 110 gallons/day) corresponds to a hole 1.8 mm (0.07 in.) in diameter. These calculated hole sizes are smaller than the "standard" hole size considered in Section 2.2.3.3.6. On the basis of these two calculations, the "standard" hole size used for calculation of leakage through FML holes is slightly conservative. It is interesting to note that the backcalculated hole sizes correspond almost exactly to the "small" hole size given in Table 2.2-16.

Eventually, waste was placed in the areas drained by LDCRS outlets 1 through 5. The flow rate of liquids draining through the LDCRS was monitored at the outlets. Initially, a high rate of liquid flow was observed (0.5 to 1.0 liter/minute (200 to 400 gallons/day)) for all five outlets. After 30 days, the flow rates had gradually decreased to a total flow rate of 0.25 liter/minute (100 gallons/day) for all five outlets. The observed leakage from outlets 2 and 4 after 30 days were 0.08 and 0.10 liter/minute (29 and 37 gallons/day), respectively. This leakage rate is approximately an order of magnitude less than the rate which was observed during the ponding test. The rate of leakage was lower than during the ponding test because the hydraulic head acting on the top liner was lower. The observed rate of leakage was attributed to consolidation of the bottom compacted clay layer as well as continued leakage through imperfections in the top FML in the vicinity of outlets 2 and 4.

After placement of the waste in the area drained by outlets 1 through 5, but prior to the placement of waste in the areas drained by LDCRS outlets 6 through 10, the landfill was inundated with rain. The LCRS outlets 6 through 10 were intentionally plugged to perform another ponding test and approximately 0.3 to 0.6 m (1 to 2 ft) of water was collected. LDCRS outlets 6 through 10 were monitored and leakage was observed only from outlet 7. The landfill was drained and the LCRS sand was removed in the area drained by outlet 7. A confined ponding test was performed in the vicinity of the point where the LCRS outlet pipe penetrated the top and bottom liners and a leak was found in the top FML boot over the pipe. The boot was repaired and another ponding test was performed; no leakage was collected during this second ponding test. The leakage through the faulty boot was approximately 1000 liters/day (250 gallons/day) under an average hydraulic head of only 0.15 m (6 in.). A leakage rate of this magnitude could be caused by a hole 4 mm (0.15 in.) in diameter.

Two conclusions can be drawn from this case study:

- First, it is important to distinguish the source of the collected liquids when evaluating lining system performance.

The rate of leakage that was initially observed during the ponding test was not all attributed to leakage through the top FML. At least part of the collected liquid was due to drainage of rainwater that had been entrapped in the LDCRS sand. Also, during placement of the waste, part of the collected liquid was attributed to consolidation of the compacted soil bottom liner.

- Second, leaks can occur at locations where pipes penetrate the landfill. These penetrations are often the most difficult part of liner installation. Nondestructive testing around penetrations is sometimes impossible. Therefore, such connections should always be carefully checked.

### 2.5.4 Case Study - Two Landfills in North Central U.S.

Two double-lined landfill units were constructed in 1985 and 1986 in the North Central U.S. for disposal of solid hazardous waste. The site of the units is located in a region that experiences about 0.7 m (2.5 ft) of rainfall annually. The ground-water elevation at the site is apparently below the elevation of the bottoms of the two units. The depths of the units vary and range from about 9 to 15 m (30 to 50 ft). The depth of the units depends primarily on the depth at the site of a native clay deposit. The two landfill units will be referred to as Unit 1 and Unit 2.

Lining system details for Unit 1 and Unit 2 are shown in Figure 2.5-3. The most significant difference between the lining systems at the two units is that the top liner at Unit 1 is a single FML, while the top liner at Unit 2 is a composite made up of an upper FML component and a 1.5-m (5-ft) thick lower compacted soil component. In both units, the LDCRS drainage media are synthetic drainage nets, so storage of liquid in the LDCRS (due to capillary tension) is negligible. The lining systems for the two units extends all the way up the 2H:1V side slopes with the top and bottom liner FMLs anchored into the same trench at the top of the slopes. In both units, the LCRS above the top liner is composed of sand on the bottom of the units and synthetic drainage nets on the side-slopes. Quality assurance of lining system installation was carried out by the owner, with quality control of FML installation performed by the FML installer.

Information on the quantities of liquid collected in the LDCRS sumps of the two units are summarized in Table 2.5-2. Referring first to Unit 1, it can be observed that the average quantity of liquid collected in the LDCRS sump is on the order of 5 Ltd (gpad) and the maximum quantity collected is on the order of 40 Ltd (gpad). Since Unit 1 uses synthetic drainage nets in the LDCRS (which result in negligible liquids storage due to LDCRS capillary tension) and an FML top liner (with no compacted soil component), and since the groundwater elevation is reported to be below the elevation of the LDCRS, the liquid in the LDCRS sump is assumed to be due to leakage through the top liner. A rate of top liner leakage of 5 Ltd (gpad) for Unit 1 corresponds to a single FML hole in the Unit 1 top liner (over the entire 14 acres) of about 3 mm (0.04 in.) diameter (assuming a hydraulic head on the top FML of 0.03 m (0.1 ft)).

From Table 2.5-2 it can be seen that the quantities of liquid collected in Unit 2 are on the order of ten times larger than the quantities collected in Unit 1. A significant part of this difference in leakage quantities can be explained by consolidation of the compacted soil component of the composite top liner. (This may be partially supported by the fact that GCMS priority pollutant scans of liquids collected in the LDCRS sump in December 1986 indicated that the collected liquids "contained no organic constituents" and had heavy metal concentrations "consistent with local background levels".) On the other hand, it may not be legitimate to attribute all of the collected liquids to consolidation of the compacted soil. In addition, it does not seem likely that the liquid is due to leachate migration through the top liner since the leachate would be required to migrate through 1.5 m (5 ft) of clay, which would be unlikely in the short elapsed time since Unit 2 was put into operation. To understand the source of all of the liquids collected in the LDCRS sump of Unit 2 will require a very careful review of the design, construction and operation of the unit. As pointed out in Section 2.5.1, it is only through a very careful review of all of the factors affecting each unit that the leakage quantities collected can be fully understood.

### 2.5.5 Case Studies - Surface Impoundments in East Central and South West U.S.

These case studies provide data on the quantity of liquids in the LDCRS sumps of surface impoundment units at two sites. The studies also provide data on lining system design, but contain no data on the construction quality assurance, operation or site hydrogeology of the units. Therefore, the information presented in these studies should be considered as preliminary until more extensive documentation becomes available.

#### 2.5.5.1 Surface Impoundments in East Central U.S.

Two surface impoundment units were constructed around 1985 in the East Central U.S. for disposal of liquid waste. These units are each 2000 $m^2$ (0.5 acre) in size and the liquid depth is approximately 6 m (20 ft). No data is available on the ground-water elevation at the site.

The lining system is the same for both of these units and details are shown in Figure 2.5-4a. Both of these units have double liner systems. The top liner consists of a 1.5 mm (60-mil) thick HDPE FML and is underlain by a LDCRS composed of a 0.3 m (1-ft) thick layer of clean sand on the bottom of the unit and geonets on the side slopes. The LDCRS is in turn underlain by a composite bottom liner comprised of a FML placed on top of compacted soil layer. No data is available on the quality assurance of lining system installation.

During the first 6 to 8 months of unit operation, the quantity of liquids removed weekly from LDCRS sumps ranged approximately from 60 to 120 liters (15 to 30 gallons). The calculated average quantity of liquids collected in the LDCRS sump is on the order of 6 Ltd (6 gpad). During the 6-8 month period, the quantities of liquids removed decreased. After this period to date, liquids were no longer detected at LDCRS sumps. The liquids removed from LDCRS sumps were analyzed for chemical constituents. The liquid total organic constituents (TOC) concentration ranged from 10 to 40 parts per million (ppm) during this 6-8 month period. The TOC of the pond influent during the same period averaged about 400 ppm. The quality of the liquids removed was reported to have improved during this 6-8 month period.

The nature of the LDCRS granular drainage medium is unknown and no data is available on ground-water elevation. Therefore, the conclusions presented hereafter can only be preliminary and can only be made by making a number of assumptions. The discussion which follows assumes that the LDCRS drainage medium consists of sand and that ground water table is below the LDCRS.

The relatively large quantities of liquids removed in LDCRS sumps at the beginning of 6-8 month period may be attributed to construction water (i.e., rainwater or other) draining slowly toward the LDCRS sumps. Drainage continued in the LDCRS drainage medium up to the point where the capillary tension in the pore space of the drainage medium became greater than gravity forces. After the drainage period, no liquids were observed in LDCRS sumps because any leakage through the top liner would be held by capillary tension in the pore space of the sand. As a result, drainage would not occur until the sand has absorbed enough liquids to fill much of its pore space. As indicated in Section 2.6, the time to fill this pore space can be as large as several years for a sand with $k_d = 10^{-4}$ m/s ($10^{-2}$ cm/s), even for relatively large top liner leakage rates such as 100 Ltd (100 gpad). Therefore, if small amounts of leakage are occurring, it is probably still being held by capillary tension in the LDCRS drainage medium.

The TOC of the liquids removed from LDCRS sumps was significantly lower than the TOC of the pond influent. This further supports that the liquids collected in the LDCRS sumps during the initial 6 - 8 month period were primarily due to drainage of water entrapped in the LDCRS drainage medium during construction. The relatively minor amount of TOC in the liquids removed may have been caused by spills from construction equipment or by a small leakage through the top liner that diluted with the larger amount of construction water draining within the LDCRS drainage medium to the LDCRS sumps.

Continued monitoring of the LDCRS is strongly encouraged to verify the conclusions drawn above for these units.

### 2.5.5.2 Surface Impoundments in South West U.S.

Thirty-two surface impoundment units were constructed in the mid-1980's in the South West U.S. for disposal of liquid waste. For these units, only quantities of liquids in LDCRS sumps and lining system details are available.

The lining system is the same for all of these units and details are shown in Figure 2.5-4b. All of these units have double liner systems which meet EPA 1985 minimum technology standards. The top liner consists of a 2.5-mm (100-mil) thick HDPE FML and the bottom liner is a composite constructed of a 2.5-mm (100-mil) thick HDPE FML on top of a compacted soil layer. The LDCRS between the liners consists of a 4-mm (0.16 in.) thick synthetic drainage net layer connected to drainage pipes. The pipes were placed in a collector trench at the bottom of the unit. The trench was backfilled with silty, sandy gravel.

The CQA program at the site called for the FML installer to carry out ponding tests in each completed unit to search for top liner leaks. The ponding tests were conducted by filling the ponds to a level 0.6 m (2 ft) above operating pool level. It is reported that only one of the 32 ponds produced liquids in the LDCRS sump during the ponding tests. An evaluation indicated that the water in the one LDCRS sump was construction water. It is safe to say that the ponding tests demonstrated that there were no major leaks in the top liner. However, small leaks (if they were present) in the range of 20 Ltd (gpad) may have gone undetected during the ponding tests due to the storage capacity and capillary tension of the collector trench backfill.

Since the beginning of operation of the first units at this facility almost two years ago, no leakage has been reported. These reports therefore indicate that top liner leakage at the units is either negligible or very small. Since the bottom liners at these surface impoundments are composites, the detection sensitivities (defined subsequently) of the units should be very good. Therefore, the detection of zero leakage in the LDCRS sumps indicates that the top liners are not leaking (i.e., zero macroscopic defects were achieved in these installations) or, at worst, that only very small (e.g., not more than 5 to 10 gpad) amounts of top liner leakage is occurring and it is not being detected because it is evaporating out of the LDCRS sump or it is being held by capillary tension in the collector trench backfill.

Table 2.5-1. Quantities of liquid collected and removed from LDCRS sump at units cited in the Institute of Chemical Waste Management Report [1986]

| Code | Type of Land Disposal Unit | Surface Area (acres) | Date Construction Completed | LDCRS Drainage Medium | Quantity of Liquids Collected and Removed from LDCRS during Active Life | |
|---|---|---|---|---|---|---|
| | | | | | Typical Average gallons/acre/day | Typical Maximum gallons/acre/day |
| A | Landfill | 9 | May 1985 | Sand, 1 ft. thick | 3.5 | 15.4 |
| B | Landfill | 11 | March 1986 | Sand, 1 ft. thick | 1.5 | 2.1 |
| C | Landfill | 3 | September 1985 | Net, 2 layers | 6.7 | 14.2 |
| D | Landfill | 2.7 | 1986 | Bottom: Sand, 1 ft thick<br>Slopes: Net, 1 layer | 18.9 | 168 |
| A | Surface Impoundment | 1.3 | June 1986 | Net, 1 layer | | 35.9 |

Note: 1 - Details of double liner systems shown on Figure 2.5-1.

Table 2.5-2. Quantities of liquid collected and removed from LDCRS sump at two landfill units in North Central U.S.

| Code | Type of Land Disposal Unit | Surface Area (acres) | Date Construction Completed | LDCRS Drainage Medium | Top Liner | Quantity of Liquids Collected and Removed from LDCRS during Active Life | |
|---|---|---|---|---|---|---|---|
| | | | | | | Typical Average gallons/acre/day | Typical Maximum gallons/acre/day |
| 1 | Landfill | 14 | Summer 1985 | Net, 1 layer | FML | 5 | 40 |
| 2 | Landfill | 9 | Summer 1986 | Net, 2 layers | Composite (FML, clay) | 100 | 500 |

Note: 1 - Details of double liner systems shown on Figure 2.5-3.

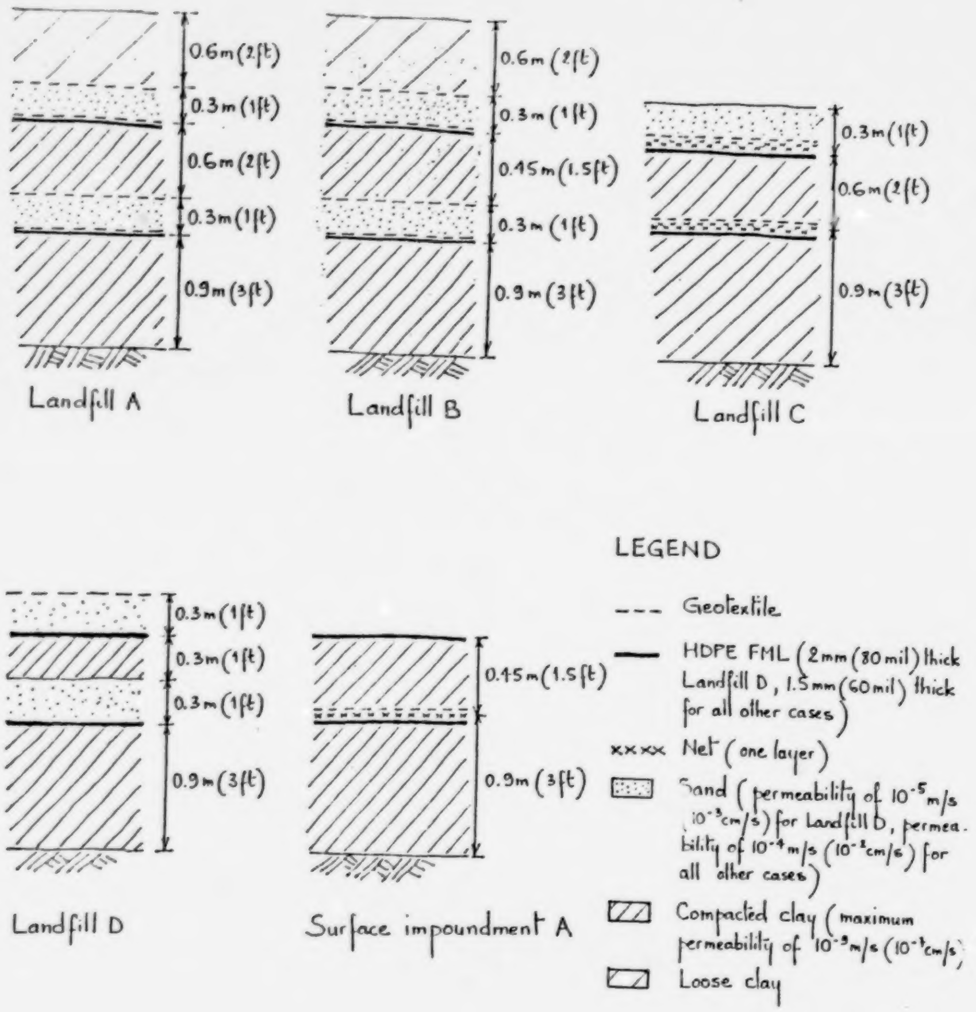


Figure 2.5-1. Details of lining system from units cited in Institute of Chemical Waste Management Report [1986].

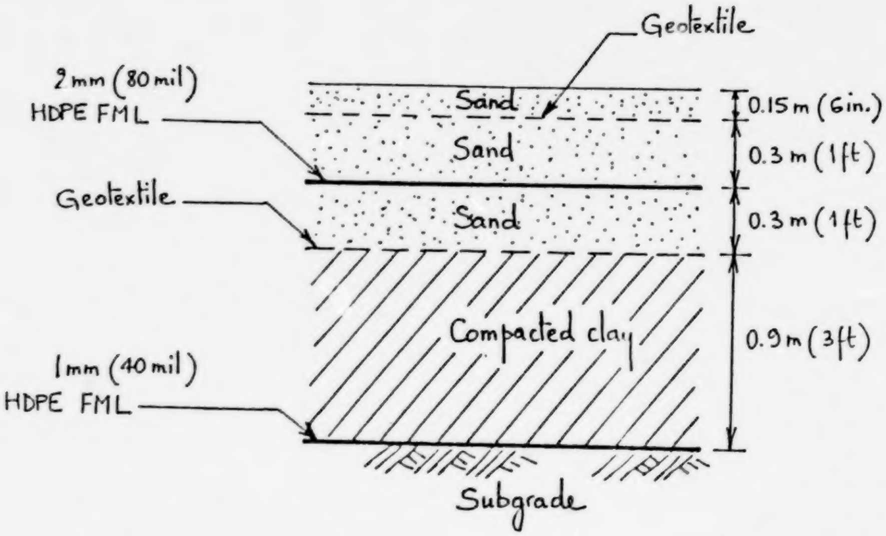


Figure 2.5-2. Details of lining system for case study of landfill in South East U.S.

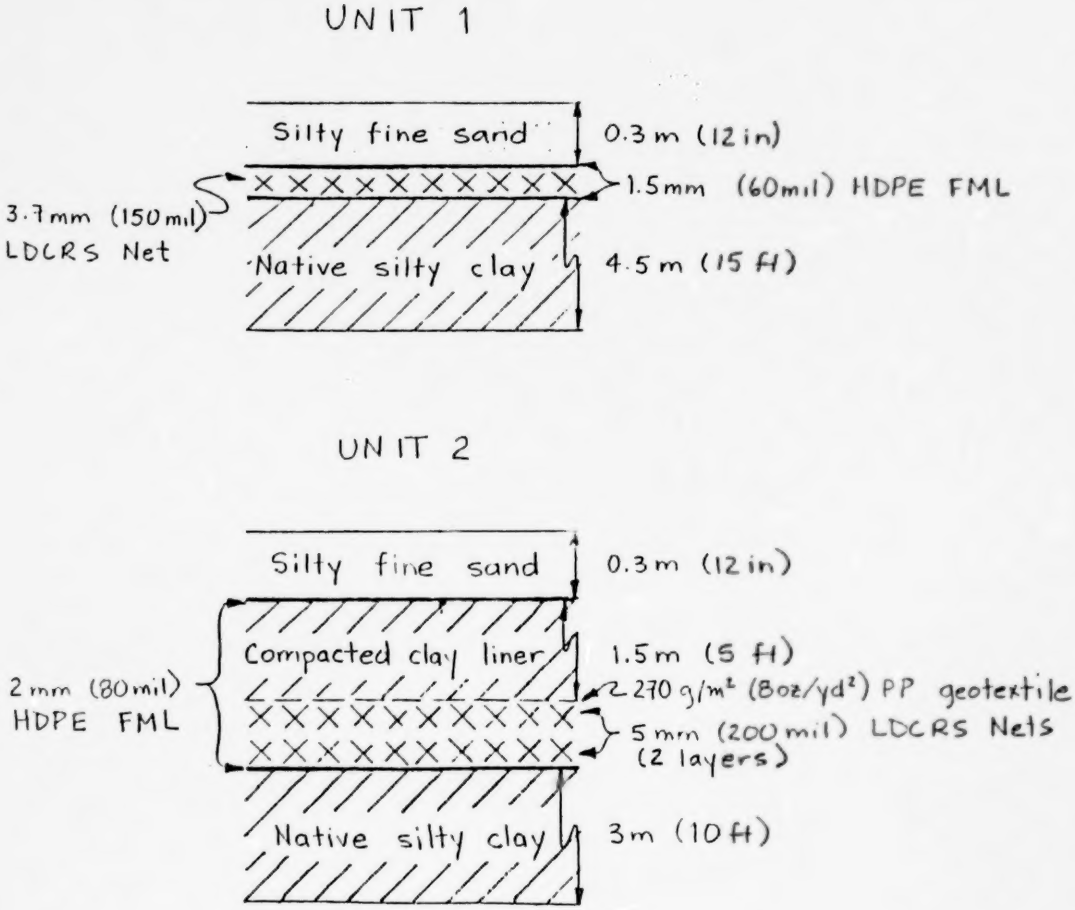
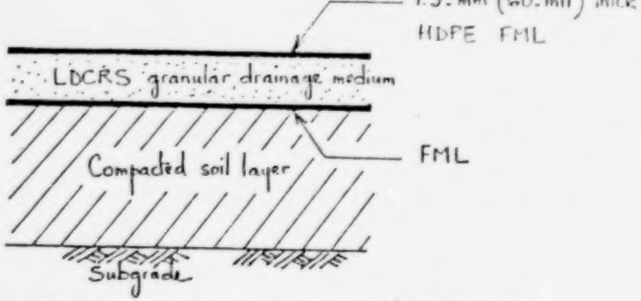
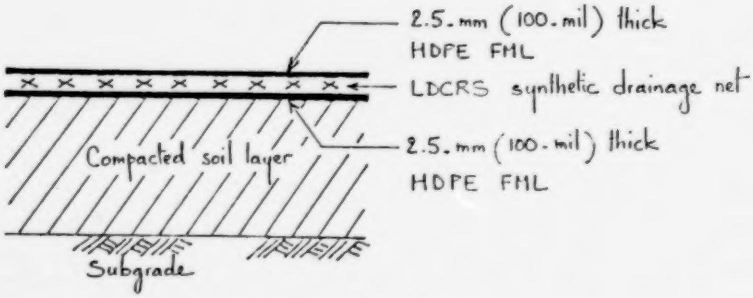


Figure 2.5-3. Details of lining systems for case study of two landfills in North Central U.S.



a- Surface impoundments in East Central U.S.



b- Surface impoundments in South West U.S.

Figure 2.5-4. Details of lining systems for case studies of surface impoundments in East Central and South West U.S.

## 2.6 ANALYSES OF THE FUNCTIONING OF LEAK DETECTION SYSTEMS

### 2.6.1 Introduction

### 2.6.1.1 Purpose

The purpose of Section 2.6 is to provide insight into the factors affecting the performance of leak detection, collection, and removal systems ("leak detection systems"). This is extremely important because several of the main technical areas addressed in the proposed Liner/Leak Detection Rule are dependent on the performance of the leak detection systems. These areas are listed below:

- Performance criteria for leak detection systems, which are discussed in Section 2.7.
- Design specifications for leak detection systems, which are discussed in Section 2.8.
- Action Leakage Rate (ALR), which is discussed in Section 2.9.
- Response Action Plan (RAP), which is discussed in Section 2.10.

A thorough understanding of the functioning of leak detection systems is necessary to establish the elements of the proposed Liner/Leak Detection Rule in the four technical areas listed above.

### 2.6.1.2 Overview of Leak Detection System Functioning

When leakage occurs through the top liner, it takes some time before the leakage is detected because of the following three reasons (see Figure 2.1-3):

- The leakage that impinges into the leak detection system may be retained by capillarity in the drainage medium during an initial period of time, instead of flowing to the sump. Then, flow takes place when some portion of the leak detection system becomes saturated. The size (thickness and width) of this saturated portion depends on the size of leak, initial capillary tension in the leak detection system, and porosity and thickness of the leak detection system drainage medium.

- Flow to a drain will be further delayed if the leak detection system is underlain by a compacted soil bottom liner. As shown in the Notice of Availability of Data on "Performance of Bottom Liners at Double-Lined Landfills and Surface Impoundments" [USEPA, 1987], compacted soil bottom liners will permit on the order of 100 Ltd (gpad) of leachate flow into and through the bottom liner (under steady-state conditions). In contrast, composite bottom liners having a FML upper component will permit less than 1 Ltd (gpad). In the remainder of this document, it will be assumed that the bottom liner is a composite with a FML upper component and a compacted soil bottom component. Since leakage into composite bottom liners is extremely small under almost all circumstances [USEPA, 1987], the simplifying assumption will be made in subsequent calculations that composite bottom liners are impermeable.

- Finally, once flow begins, it takes time for the leakage to flow from the location of the leak to the sump.

From the above discussion (and assuming an impermeable composite bottom liner), it is clear that two phases of the functioning of a leak detection system should be investigated: (i) the period of initiation of the flow; and (ii) the steady-state flow.

The parameters which govern the functioning of a leak detection system are:

- Parameters related to the top liner: leakage rate through the top liner, type of leak.

- Parameters related to the leak detection system: characteristics governing storage capacity (size of the voids, porosity, thickness, initial water content); and characteristics governing the flow (hydraulic conductivity, thickness, slope, length of flow path).

- Parameters related to the bottom liner: leakage rate through the bottom liner, type of leak.

The studies presented in Section 2.6 will provide information regarding the influence of these parameters on the performance of leak detection systems. Other pertinent data are provided in Sections 2.2 (on leakage through top liners), Section 2.4 (on characteristics of leak detection systems), and Section 2.5 (on performance of leak detection systems). Pertinent information on the performance of bottom liners and their influence on the functioning of leak detection systems may be found in USEPA [1987].

The functioning of a leak detection system may also be significantly affected by sources of water other than leakage, such as:

- Rainwater entrapped in the leak detection system during construction of the unit.

- Water used to compact the low-permeability compacted soil component, if any, of the top liner.

- Water flowing into the waste management unit as a result of a rise in the ground water table elevation.

Sources of liquid (water) other than leakage through the top liner affect only the Response Action Plan (RAP) among the four technical issues listed in Section 2.6.1.1. Therefore the effect of sources of liquid other than leakage through the top liner will be discussed only in Section 2.10.

### 2.6.1.3 Definitions

Terms and phrases specific to the functioning of leak detection systems are used in Section 2.6 and need to be defined.

### 2.6.1.3.1 Leakage

In this context, the term leakage is used for "leakage through the top liner". There is leakage as soon as leachate has just passed through the top liner and impinges into the leak detection system.

### 2.6.1.3.2 Time of Initial Leakage

The "time of initial leakage" is the time when leakage first occurs. A "time of initial leakage" can be considered for each specific cause of leakage. For example, the time of initial leakage related to a given crack through the top liner may be one year after the beginning of the operation of the hazardous waste land disposal unit if this is the time it took the crack to develop, plus the time for leakage to go through the crack.

### 2.6.1.3.3 Initial Detection Time

The "initial detection time" is the time required to detect leakage after it first occurs (i.e., the time difference between the time of initial leakage and the time when this leakage appears at the sump).

### 2.6.1.3.4 Leak Detection Time

The "steady-state leak detection time" (simply called the "leak detection time") is the time between when a drop of leakage enters the leak detection system (i.e., the time when this drop has just passed through the top liner) and the time it appears in the collector pipes or sump in a steady-state flow. This concept does not apply to the first drop of leakage; it applies to any drop of leakage after the leakage and the flow in the leak detection system have reached a steady state.

### 2.6.1.3.5 Detection Sensitivity

Detection sensitivity refers to the smallest top liner leakage rate which can be detected.

#### 2.6.1.3.6 Action Leakage Rate (ALR)

Action Leakage Rate (ALR) refers to the rate of leakage from the top liner into the LDCRS that triggers interaction between the owner or operator and the EPA Regional Administrator to determine the appropriate response action for leakage.

#### 2.6.1.3.7 Rapid and Extremely Large Leakage

Rapid and Extremely Large Leakage (RLL) is defined as the maximum design leakage rate that the leak detection system (LDCRS) can remove without exceeding a maximum fluid head (pressure) on the bottom (outside the sump) equivalent to 0.3 m (1 ft) of water.

### 2.6.1.4 Organization of this Section

Two approaches have been selected to analyze the functioning of leak detection systems: analytical and numerical.

Section 2.6.2 is devoted to an analytical study conducted by GeoServices Inc., Boynton Beach, FL and Section 2.6.3 is devoted to a study using a finite element numerical model conducted by Radian Corporation, Austin, TX.

## 2.6.2 Two-Dimensional Analytical Study

### 2.6.2.1 Introduction

#### 2.6.2.1.1 Purpose of this Section

The purpose of this section is to present an analysis of the functioning of leak detection systems which provides numerical information on key parameters involved in the leak detection rule such as:

- detection sensitivity;
- detection time;

- action leakage rate; and
- rapid and extremely large leak.

This study was performed by GeoServices Inc. Consulting Engineers of Boynton Beach, Florida, and is described in [Giroud et al., 1987b].

### 2.6.2.1.2 Approach

The approach chosen was to conduct an analytical study. Such studies can provide results that numerical methods often do not provide because of inherent limitations. Also, because of the complexity of numerical methods, more time and effort are usually spent in operating the method than in studying the physical problem, discussing the validity of the results, and drawing practical conclusions.

### 2.6.2.1.3 Organization of this Section

Section 2.6.2 is organized as follows:

- Section 2.6.2.2 presents the assumptions made for the analytical study.
- Section 2.6.2.3 presents the analyses related to steady flow, including an evaluation of the steady-state detection time ("detection time").
- Section 2.6.2.4 presents the analysis related to flow initiation, including an evaluation of the initial detection time.
- Section 2.6.2.5 presents conclusions pertinent to important aspects of the proposed Liner/Leak Detection Rule such as: detection sensitivity, detection time, action leakage rate, and rapid and extremely large leakage.

### 2.6.2.2 Assumptions

#### 2.6.2.2.1 Assumptions Related to the Leak Detection System

- Leak Detection System Geometry

The leak detection system area considered has the following geometry (Figure 2.6-1):

- length L along the slope;
- width B across the slope (i.e., horizontal);
- depth D; and
- slope angle $\beta$.

- Leak Detection System Material

The pervious material used in the leak detection system has the following characteristics:

- porosity n; and
- saturated hydraulic conductivity, $k_d$.

#### 2.6.2.2.2 Assumptions Related to the Flow

- Flow Conditions

The flow conditions are as follows:

- the liner underlying the pervious material used in the leak detection system is perfectly impermeable (i.e., its hydraulic conductivity is zero and it has no holes, cracks, etc.);

- the only liquids considered in the leak detection system are those resulting from leakage through the top liner (i.e., liquids such as water entrapped during construction and ground water migrating through the bottom liner are not considered here - see Section 2.10);

- the only leakage that impinges the considered area of the leak detection system is a uniform leak that impinges the leak detection system along its higher edge; and

- leakage is collected uniformly along the lower edge of the leak detection system.

These assumptions are those of a bidimensional flow. Therefore, B can be regarded as a unit width.

- Leakage Rate

The leakage rate, Q, that impinges the leak detection system at its higher edge is expressed in units of volume per unit of time: Q ($m^3/s$, liters/day, gallons/day). This leakage rate can also be expressed as a volume per unit of area and unit of time, q (m/s, $m^3/m^2/s$, liters/1000$m^2$/day (Ltd), gallon/acre/day (gpad)), using the following relationship:

$$q = Q/(LB) \qquad \text{(Equation 2.6-1)}$$

where: q = leakage rate per unit area (often called leakage rate); Q = leakage rate; L = length of considered area in the direction of the flow; and B = width of the considered area. Recommended SI units are: q (m/s), Q ($m^3/s$), L (m), and B (m).

- Flow Mechanism

The flow mechanism is assumed to be as follows:

- When leakage first occurs (i.e., when a first drop of liquid has just passed through the top liner), liquid is first held by capillarity in the leak detection system. The volume of liquid which can be held by capillarity depends on the opening size of the material used in the leak detection system and the moisture content of the leak detection system material before leakage occurs.

- Some time after the beginning of leakage, enough liquid has passed through the top liner to provide all the volume which can be held by capillarity. This point is the beginning of steady flow of liquid in the leak detection system.

These two mechanisms are discussed hereafter. It is more convenient to discuss steady state first (Section 2.6.2.3), and, then, to discuss the initial period when liquid is held by capillarity (Section 2.6.2.4).

### 2.6.2.3 Steady-State Flow

#### 2.6.2.3.1 Introduction

- Scope

In this section, it is assumed that the flow in the leak detection system is in a steady-state condition. This becomes possible when a portion of the leak detection system which is continuous between the location of the leak and the sump has become saturated as a result of progressive accumulation of water held by capillarity.

- Organization of this Section

First, Section 2.6.2.3.2 presents an evaluation of the "steady-state leak detection time" (also called "leak detection time" or "detection time"). Then, Section 2.6.2.3.3 discusses the steady-state flow capacity of the leak detection system. The flow capacity will be used in Section 2.10 to determine the rapid and extremely large leak (RLL) that would generate excessive head on the bottom liner.

### 2.6.2.3.2 Steady-State Leak Detection Time

- Darcy's Equation

Steady-state flow is governed by Darcy's equation. According to Darcy's equation, the apparent velocity of flow in a saturated pervious medium, such as the leak detection system material, is given by:

$$v = Q_d/A = k_d\ i \qquad \text{(Equation 2.6-2)}$$

where: $Q_d$ = flow rate in the leak detection system; $A$ = cross-section area of the leak detection system perpendicular to the flow; $k_d$ = hydraulic conductivity of the leak detection system material; and $i$ = hydraulic gradient (= head loss/length of flow path).

For saturated free flow on a slope, the head is provided by the slope only and, since for a length of flow path L, the altitude drops by $L \sin\beta$, the hydraulic gradient is:

$$i = \sin\beta \qquad \text{(Equation 2.6-3)}$$

where: $\beta$ = slope angle.

- Flow Velocity

The velocity v given by Darcy's equation (Equation 2.6-2) is the apparent velocity, i.e., the flow rate divided by the cross section of the flow. In reality, liquid does not flow over the entire cross section of the soil but only in the pores of the soil. Therefore, liquid travels from one point to another with a velocity given by:

$$v' = v/n = k_d\ i/n \qquad \text{(Equation 2.6-4)}$$

By combining Equations 2.6-2 and 2.6-4, velocity of flow on a slope is given by:

$$v' = k_d \sin\beta/n \qquad \text{(Equation 2.6-5)}$$

### - Time for Detecting Leakage

The "steady-state leak detection time", simply called "leak detection time", is the time leakage travels from the location of the leak to a collector pipe or sump in a steady-state condition. In the model discussed above, this time is the time for leakage to travel along the slope, from the higher edge to the lower edge of the leak detection system. Using Equation 2.6-5, it appears that this time is given by:

$$t_d = nL/(k_d \sin\beta) \qquad \text{(Equation 2.6-6)}$$

where $t_d$ = leak detection time; $n$ = porosity of the leak detection system material; $L$ = length of the flow path (i.e., distance between the leak in the top liner and the sump); $k_d$ = hydraulic conductivity of the leak detection system material; and $\beta$ = slope of the leak detection system. Recommended SI units are: $t_d$ (s), $L$ (m), and $k_d$ (m/s); $n$ and $\beta$ are dimensionless.

Equation 2.6-6 becomes:

$$\begin{aligned} t_d &= n\,L/(86{,}400\;k_d \sin\beta) \\ &= 1.16 \times 10^{-5}\; n\,L/(k_d \sin\beta) \qquad \text{(Equation 2.6-7)} \end{aligned}$$

with: $t_d$ (days), $L$ (m), and $k_d$ (m/s)

$$\begin{aligned} t_d &= n\,L/(2835\;k_d \sin\beta) \\ &= 3.5 \times 10^{-4}\; n\,L/(k_d \sin\beta) \qquad \text{(Equation 2.6-8)} \end{aligned}$$

with: $t_d$ (days), $L$ (ft), and $k_d$ (cm/s)

For a porosity $n = 0.3$ of the leak detection system and a 2% slope, the above equations become:

$$t_d = 1.74 \times 10^{-4}\; L/k_d \qquad \text{(Equation 2.6-9)}$$

with $t_d$ (days), $L$ (m), and $k_d$ (m/s)

$$t_d = 5.29 \times 10^{-3}\; L/k_d \qquad \text{(Equation 2.6-10)}$$

with $t_d$ (days), $L$ (ft), and $k_d$ (cm/s)

- Chart and Table for Leak Detection Time

Using Equation 2.6-9, a chart giving leak detection time has been established and is presented in Figure 2.6-2. A logarithmic scale was selected, although it is not necessary, to match with the charts related to flow initiation (presented in the next section) where a logarithmic scale is appropriate.

Table 2.6-1 has been established from Equation 2.6-9 or Figure 2.6-2. This table gives leak detection times as a function of: (i) the distance between the leak and the collector pipe or sump; and (ii) the hydraulic conductivity of the leak detection system material.

Table 2.6-1 and the chart presented in Figure 2.6-2 show that leak detection time (i.e., steady-state leak detection times) on the order of one day or less can be obtained using a leak detection system material hydraulic conductivity of $10^{-2}$ m/s (1 cm/s) or more.

### 2.6.2.3.3 Leak Detection System Capacity

- Leak Detection System Hydraulic Transmissivity

According to Darcy's equation (Equation 2.6-2), the flow rate in the leak detection system can be expressed as follows:

$$Q_d/B = k_d\ i\ D = k_d\ D \sin \beta \qquad \text{(Equation 2.6-11)}$$

where: $Q_d$ = flow rate in the leak detection system; $B$ = width of the considered section of the leak detection system perpendicular to the flow; $Q_d/B$ = flow rate per unit width; $k_d$ = leak detection system material hydraulic conductivity; $i$ = hydraulic gradient; $D$ = leak detection system thickness; and $\beta$ = slope of the leak detection system. Recommended SI units are: $Q$ ($m^3/s$), $B$ (m), $Q/B$ ($m^2/s$), $k_d$ (m/s), and $D$ (m); $\beta$ and $i$ are dimensionless.

Equation 2.6-11 can be written as follows:

$$Q_d/B = \theta_d\ i = \theta_d \sin \beta \qquad \text{(Equation 2.6-12)}$$

where $\theta_d$ is the hydraulic transmissivity of the leak detection system, defined by:

$$\theta_d = k_d D \qquad \text{(Equation 2.6-13)}$$

The recommended SI unit for $\theta_d$ is m²/s.

According to Equation 2.6-12, the capacity of the leak detection system is governed by its hydraulic transmissivity. Equation 2.6-13 shows that, for a given leak detection system material hydraulic conductivity, $k_d$, the capacity of the leak detection system is governed by its thickness D.

- Relationship Between Leakage Rate and Transmissivity

The flow rate, $Q_d$, in the leak detection system is zero at the higher edge of the considered model (Figure 2.6-1) and is equal to the leakage rate, Q, at the lower edge of the considered model. In other words, the maximum value of the flow rate $Q_d$ in the leak detection system is:

$$Q_{d\,max} = Q \qquad \text{(Equation 2.6-14)}$$

According to Equation 2.6-1, Equation 2.6-14 can be rewritten as:

$$Q_{d\,max} = q\,L\,B \qquad \text{(Equation 2.6-15)}$$

Combining Equation 2.6-11, 2.6-13 and 2.6-15, the following relationships can be established between the leakage rate through the top liner and the characteristics of the leak detection system:

$$q\,L = k_d\,D \sin\beta \qquad \text{(Equation 2.6-16)}$$

$$q\,L = \theta_d \sin\beta \qquad \text{(Equation 2.6-17)}$$

where: q = leakage rate per unit area (also called "leakage rate") through the top liner; L = flow path (i.e., distance between the leak and the sump); $k_d$ = hydraulic conductivity of the leak detection system material; $D$ = thickness of the leak detection system; $\beta$ = slope of the leak detection system; and $\theta_d$ = hydraulic transmissivity of the leak detection system. Recommended SI units are: $q$ (m/s), $L$ (m), $k_d$ (m/s), $D$ (m), and $\theta_d$ (m²/s); $\beta$ is dimensionless.

- Required Capacity of the Leak Detection System

If the transmissivity (i.e., thickness x hudraulic conductivity) of the leak detection system is not sufficient to convey the leakage through the top liner, hydraulic head builds up in the leak detection system. Leakage into the ground increases as a result of increased head on the bottom liner, which should absolutely be avoided. Therefore, the hydraulic transmissivity and the thickness of the leak detection system must have the following minimum values derived from Equation 2.6-16 and 2.6-17:

$$\theta_d \geq q\ L/\sin\beta \qquad \text{(Equation 2.6-18)}$$

$$D \geq q\ L/k_d \sin\beta \qquad \text{(Equation 2.6-19)}$$

(Notation defined after Equation 2.6-17.)

Equations 2.6-18 and 2.6-19 can be used with any compatible systems of units, including the recommended SI units: $\theta_d$ (m²/s), $q$ (m/s), $L$ (m), $D$ (m), and $k_d$ (m/s).

Equation 2.6-18 can be written as follows with various units:

$$\theta_d \geq 1.16 \times 10^{-11}\ q\ L/\sin\beta \qquad \text{(Equation 2.6-20)}$$

with: $\theta$ (m²/s), $q$ (Ltd) and $L$ (m)

$$\theta_d \geq 3.3 \times 10^{-12}\ q\ L/\sin\beta \qquad \text{(Equation 2.6-21)}$$

with: $\theta_d$ (m²/s), $q$ (gpad), $L$ (ft)

Equation 2.6-19 can be written as follows with various systems of units:

$$D \geq 1.16 \times 10^{-11}\ q\ L/(k_d \sin \beta) \qquad \text{(Equation 2.6-22)}$$
with: D (m), q (Ltd), L (m), $k_d$ (m/s)

$$D \geq 1.16 \times 10^{-9}\ q\ L/(k_d \sin \beta) \qquad \text{(Equation 2.6-23)}$$
with: D (ft), q (gpad), L (ft), $k_d$ (cm/s)

In the special case of a 2% slope ($\sin \beta = 0.02$), Equations 2.6-18 and 19 become:

$$\theta_d = 5.8 \times 10^{-10}\ q\ L \qquad \text{(Equation 2.6-24)}$$
with: $\theta$ (m²/s), q (Ltd), L (m)

$$\theta_d = 1.65 \times 10^{-10}\ q\ L \qquad \text{(Equation 2.6-25)}$$
with: $\theta_d$ (m²/s), q (gpad), L (ft)

$$D = 5.8 \times 10^{-10}\ q\ L/k_d \qquad \text{(Equation 2.6-26)}$$
with: D (m), q (Ltd), L (m), $k_d$ (m/s)

$$D = 5.4 \times 10^{-8}\ q\ L/k_d \qquad \text{(Equation 2.6-27)}$$
with: D (ft), q (gpad), L (ft), $k_d$ (cm/s)

- Tables

Table 2.6-2 was established using Equations 2.6-24 and 2.6-25. This table gives the required hydraulic transmissivity for various leakage rates. In order to select the minimum hydraulic transmissivity a leak detection system should have, it is recommended that a leakage rate of 10,000 Ltd (gpad) be considered. This value of leakage rate provides a factor of safety on the order of 10 vis a vis the "rapid and extremely large leak" (RLL) which can be on the order of 1,000 Ltd (gpad). Factors of safety of 10 are typical in hydraulic designs for critical projects.

Considering a leakage rate of 10,000 Ltd (gpad), Table 2.6-2 shows that a minimum hydraulic transmissivity, $\theta_d$, on the order of $5 \times 10^{-4}$ m²/s should be considered for a leak detection system. According to Equation 2.6-13, such a hydraulic transmissivity can be achieved by many combinations of hydraulic conductivity, $k_d$, and thickness, D, of the leak detection system such as:

- $k_d = 10^{-1}$ m/s (10 cm/s) and D = 5 mm; and

- $k_d = 10^{-2}$ m/s (1 cm/s) and D = 50 mm.

Therefore, from a practical standpoint, two types of leak detection systems should be considered:

- synthetic leak detection systems with $k_d > 10^{-1}$ m/s (10 cm/s) and D > 5 mm (0.2 in.); and

- granular leak detection systems with $k_d > 10^{-2}$ m/s (1 cm/s) and D > 300 mm (12 in.).

In the latter case, the technical requirement of 50 mm has been replaced by 300 mm for two reasons: (i) placement of a soil layer much less than 300 mm (1 ft) is not practical; and, (ii) the construction equipment used to spread granular materials should not operate on top of the FML component of the bottom liner unless the FML is covered by a protective layer approximately 300 mm (1 ft) thick.

Table 2.6-3 has been established using Equations 2.6-26 and 2.6-27, for a leakage rate of 10,000 Ltd (gpad). This table gives required leak detection system thicknesses and illustrates that with a slope of 2%, a thickness on the order of 5 mm (0.2 in.) is required with a hydraulic conductivity of 0.1 m/s (10 cm/s), while a thickness on the order of 50 mm (2 in.) is required with a hydraulic conductivity of 0.01 m/s (1 cm/s).

### 2.6.2.4 Initiation of Flow

#### 2.6.2.4.1 Introduction

- Scope

In this section, it is assumed that leakage has just occurred through the top liner and has just entered the leak detection system. The purpose of this section is to evaluate the amount of liquid held by capillarity in the leak detection system before the flow reaches a steady state.

- Organization

First, Section 2.6.2.4.2 presents an evaluation of the amount of liquid held by capillarity in the leak detection system, and the rate at which this amount increases, which governs the initial leak detection time. Then, Section 2.6.2.4.3 discusses the limits of this mechanism in case of large leakage rates (at which point, flow is governed by Darcy's equation).

### 2.6.2.4.2 Retention by Capillarity

- Assumption

In this study we assume that the zone where liquid is held by capillarity covers the entire area considered (i.e., L x B) over a height equal to: (i) the capillary rise, h, for the considered pervious medium, if h is smaller than the thickness D of the leak detection system; or (ii) the entire thickness D, if the capillary rise, h, is larger than D.

- Capillary Rise

The capillary rise in a tube is given by the classic Jurin's equation:

$$h_c = 4T/(\rho\ g\ d_t) \qquad \text{(Equation 2.6-28)}$$

where: $h_c$ = capillary rise; T = capillary tension (which depends on the liquid and, to a lesser extent, on the material in contact); $\rho$ = [illegible]

g (9.81 m/s²), and $d_t$ (m). [illegible]

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For water:

$$T = 7.64 \times 10^{-2} \text{ N/m}$$

$$\rho = 1000 \text{ kg/m}^3$$

In order to use Jurin's equation for granular drainage materials, we assume that the soil pore diameter governing capillary rise is equal to the $d_{10}$ particle size of the soil (i.e., the soil particle size which is larger than 10% by weight of the soil particles). In addition, particle size can be linked to hydraulic conductivity using Hazen's equation:

$$k_d = C\,(d_{10})^2 \qquad \text{(Equation 2.6-29)}$$

where: $C$ = Hazen's coefficient, equal to $10^4 \text{ m}^{-1}\text{s}^{-1}$ ($10^2 \text{ cm}^{-1}\text{s}^{-1}$).

A relationship between capillary rise and hydraulic conductivity can therefore be obtained by combining Equations 2.6-28 and 2.6-29, and assuming $d_t = d_{10}$, as indicated above:

$$h_c = 4T/(\rho\, g\, \sqrt{k_d/C}) \qquad \text{(Equation 2.6-30)}$$

A similar equation related to non-granular drainage materials could be obtained by combining Jurin's equation with Poiseuille's equation (instead of Hazen's equation). (Kozeny's approach could be used to establish a relationship between the two equations.) In the mean time Equation 2.6-31 will be used for all types of drainage materials.

Replacing $T$ and $\rho$ by their values for water ($T = 7.64 \times 10^{-2}$ N/m, $\rho = 1000 \text{ kg/m}^3$) and $g$ and $C$ by their values ($g = 9.81 \text{ m/s}^2$, $C = 10^4 \text{ m}^{-1}\text{ s}^{-1}$), the following water capillary rise is obtained:

$$h_c = 3.1 \times 10^{-3}/\sqrt{k_d} \qquad \text{(Equation 2.6-31)}$$

(Note: this equation is valid only for $k_d$ in m/s and gives $h_c$ in m)

Typical values of capillary rise as a function of hydraulic conductivity are given in Table 2.6-4.

- Volume of Liquid Held by Capillarity

While liquid is being held by capillarity, it does not flow towards the sump, which delays leakage detection. Two cases should be considered regarding the volume of leakage which can be held by capillarity in the leak detection system (in addition to the volume of liquid already held by capillarity before the considered leakage starts):

- if $h_c < D$:

$$V = n\,LBh\,(S_r - S_{ro}) \qquad \text{(Equation 2.6-32)}$$

- if $h_c > D$:

$$V = n\,LBD\,(S_r - S_{ro}) \qquad \text{(Equation 2.6-33)}$$

where: $V$ = volume of liquid held by capillarity; $h_c$ = capillary rise; $D$ = thickness of leak detection system; $L$ = length of leak detection system along the slope; $B$ = width of leak detection system across the slope; $n$ = porosity of the leak detection system material; $S_r$ = degree of saturation of the capillary zone at the beginning of leak detection; and $S_{ro}$ = initial degree of saturation of the capillary zone (i.e., degree of saturation at the time of initial leakage). Recommended SI units are: $V$ ($m^3$), $h_c$ (m), $D$ (m), $L$ (m), $B$ (m); $n$, $S_r$, and $S_{ro}$ are dimensionless and less than one.

The initial degree of saturation, $S_{ro}$, depends on the water content of the leak detection system material at the beginning of the considered leakage. This water content may result from: (i) water entrapped in the leak detection system during construction; (ii) water used to compact the low-permeability soil, if any, included in the top liner; (iii) ground water seeping into the leak detection system; and (iv) previous leakage.

### - Detection Delay Caused by Capillarity

Although the liquid is being held by capillarity, it progresses toward the sump as the capillary zone grows. Therefore, if we assume that there is no free flow (i.e., no Darcy's flow), the initial detection time is the time necessary for the capillary zone to grow from the source of leakage (elevated edge of the considered drainage area as shown in Figure 2.6-1) to the sump (located at the lower edge of the considered area). This time can be obtained by dividing the volume of liquid which can be stored in the capillary zone (Equation 2.6-32 or 2.6-33) by the leakage rate (Equation 2.6-1):

$$t_i = V/(qLB) \qquad \text{(Equation 2.6-34)}$$

which leads to:

$$t_i = n\ (S_r - S_{ro})\ h_c/q \qquad (\text{if } h < D) \qquad \text{(Equation 2.6-35)}$$

$$t_i = n\ (S_r - S_{ro})\ D/q \qquad (\text{if } h > D) \qquad \text{(Equation 2.6-36)}$$

where: $t_i$ = initial detection time; $h_c$ = capillary rise; $D$ = thickness of leak detection system; $n$ = porosity of material used in leak detection system; $S_r$ = degree of saturation of the capillary zone at the beginning of leak detection; $S_{ro}$ = initial degree of saturation of the potential capillary zone; and $q$ = leakage rate. Recommended SI units are: $t_i$ (s), $h_c$ (m), $D$ (m), $q$ (m/s); $n$, $S_r$, and $S_{ro}$ are dimensionless and less than one.

By combining Equations 2.6-30 and 2.6-35, the following equation is obtained:

$$t_i = 4n\ (S_r - S_{ro})\ T/(\rho\ g\ q\ \sqrt{k_d/C}) \qquad \text{(Equation 2.6-37)}$$

Using the numerical values which were used for Equation 2.6-31, and using $n = 0.3$ and $S_r = 1$, the following equation is obtained:

$$t_i = 9.3 \times 10^{-3}\ (1 - S_{ro})/(q\ \sqrt{k_d}) \qquad \text{(Equation 2.6-38)}$$

(Note: this equation is valid only with the following units: $t_i$ (s), q (m/s), and $k_d$ (m/s).)

This equation becomes:

$$t_i = 928\ (1 - S_{ro})/(q\ \sqrt{k_d}) \qquad \text{(Equation 2.6-39)}$$

if the following units are used: $t_i$, in days; q, in liters/1000m²/day (Ltd); and $k_d$, in m/s.

Equation 2.6-38 becomes:

$$t_i = 10{,}200\ (1 - S_{ro})/(q\ \sqrt{k_d}) \qquad \text{(Equation 2.6-40)}$$

if the following units are used: $t_i$, in days; q, in gallons/acre/day (gpad); and $k_d$ in cm/s.

- Establishment of a Chart

Equation 2.6-38 can be represented by a family of hyperbolas giving the initial detection time $t_i$ as a function of the leakage rate q, each hyperbola being related to a given value of the hydraulic conductivity, $k_d$, of the leak detection system. A practical way to represent the relationship between detection time and leakage rate is to use a log-log scale (Figure 2.6-3): as a result, the family of hyperbolas becomes a family of straight lines at 45°. (Note that Figure 2.6-3 has been established for an initial degree of saturation, $S_{ro}$, equal to zero; in other words, Figure 2.6-3 is related to the case where the leak detection system is dry when leakage begins.)

It is important to notice that the family of curves is bounded by the curve which is related to the hydraulic conductivity that gives a capillary rise equal to the thickness of the leak detection system. For example, if the thickness of the leak detection system is D = 0.3 m (1 ft), all curves in Figure 2.6-3 for $k_d < 10^{-4}$ m/s ($10^{-2}$ cm/s) merge with the curve for $k_d = 10^{-4}$ m/s because the capillary rise for $k_d = 10^{-4}$ m/s is 0.3 m according to Table 2.6-4. In other words, any calculation done with $k_d < 10^{-4}$ m/s ($10^{-2}$ cm/s) gives the same value of the initial detection time than a calculation conducted with $k_d = 10^{-4}$ m/s ($10^{-2}$ cm/s), if the thickness of the leak detection system is D = 0.3 m (1 ft).

Portions of the curves shown in Figure 2.6-3 are not valid because they are incompatible with Darcy's flow for the reasons explained in Section 2.6.2.4.2. It is therefore necessary to establish a more complete chart combining the effects of capillarity and Darcy's flow.

### 2.6.2.4.2 Boundaries of the Flow Mechanism

- Description of the Phenomenon

The front of the volume which is being saturated by capillarity cannot move faster than laminar flow in a saturated medium, which is governed by Darcy's equation (Equation 2.6-1). Drainage into pipes or sumps will not occur until the saturated front reaches the pipe or sump. Therefore initial detection times, $t_i$, given by Equation 2.6-37 and by the chart presented in Figure 2.6-3 are valid only if they are larger than the leak detection times, $t_d$, related to Darcy's flow and given by Equation 2.6-6. This provides a boundary to the flow mechanism, which is discussed below.

- Chart Combining Capillarity and Darcy's Flow

A refinement of the chart given in Figure 2.6-3 consists of including the times $t_d$ for Darcy's flow. According to Equation 2.6-6, such times are independent of the leakage rate q. Therefore, in the t-q chart, the Darcy's times (i.e., the detection times), $t_d$, will be represented by horizontal lines. These horizontal lines meet the 45° lines related to the capillary times (i.e., the initial detection times), $t_i$, at points defined by:

$$t_i = t_d \qquad \text{(Equation 2.6-41)}$$

The locus of these points is obtained by eliminating the hydraulic conductivity $k_d$ in the following set of equations, Equation 2.6-41 above, and Equations 2.6-6, 2.6-37, which are reproduced below:

$$t_d = nL/(k \sin\beta)$$

$$t_c = 4n\ (S_r - S_{ro})\ T/(\rho\ g\ q\ \sqrt{k/C})$$

This gives:

$$t_d = t_i = 16\ n\ C \sin\beta\ L^{-1}\ [(S_r - S_{ro})\ T/(\rho\ g\ q)]^2 \qquad \text{(Equation 2.6-42)}$$

Using the following values:

$n$ = 0.3 (porosity of material used in leak detection system),

$C$ = $10^4$ $m^{-1}$ $s^{-1}$ (Hazen's coefficient),

$\beta$ = arc tan 0.02 (bottom slope),

$S_r$ = 1 (degree of saturation in the capillary zone of the material used in the leak detection system at the beginning of leak detection),

$S_{ro}$ = 0 (initial degree of saturation in the potential capillary zone of the material used in the leak detection system),

$T$ = $7.64 \times 10^{-2}$ N/m (water capillary tension),

$\rho$ = 1000 kg/m$^3$ (density of water), and

$g$ = 9.81 m/s$^2$ (acceleration of gravity),

the following equations are obtained:

$$t_d = t_i = 5.8 \times 10^{-8}/(Lq^2) \qquad \text{(Equation 2.6-43)}$$

(with the following units: t (s), L (m), q (m/s)) or:

$$t_d = t_i = 5 \times 10^9/(Lq^2) \qquad \text{(Equation 2.6-44)}$$

(with the following units: t (days), L (m), q (liters/1000m²/day))

or:

$$t_d = t_i = 2 \times 10^{10}/(Lq^2) \qquad \text{(Equation 2.6-45)}$$

(with the following units: t (days), L (ft), q (gallons/acre/day))

where: $t_d$ = leak detection time; $t_i$ = initial detection time; L = length of flow path (distance between leak and pipe or sump); and q = leakage rate per unit area.

The above equation is the equation of dashed line AB in Figure 2.6-4 (for L = 60 m = 200 ft), and dashed lines in Figure 2.6-5. The dashed lines in these figures are the loci of points where Darcy's curves meet capillarity curves, i.e., where initial detection time equals leak detection time. This happens for large leakage rates: steady-state flow is then established immediately, i.e., without a preliminary phase with capillary retention.

- Required Hydraulic Transmissivity

A second boundary of the flow mechanism results from the fact that Darcy's flow rate is limited by the hydraulic transmissivity of the leak detection system. This is discussed in Section 2.6.2.3.3 where it is shown that this boundary is reached when the leakage rate reaches the following value derived from Equation 2.6-16:

$$q = k\ D \sin\beta/L \qquad \text{(Equation 2.6-46)}$$

Combining Equation 2.6-46 with Equation 2.6-6 (which gives the Darcy's detection time $t_d$) and eliminating $k_d$ lead to:

$$t_d = n\,D/q \qquad \text{(Equation 2.6-47)}$$

where: n = porosity of the leak detection system material; D = thickness of the leak detection system; and q = leakage rate per unit area.

Equation 2.6-47 is identical to Equation 2.6-36 which is related to capillarity over the entire thickness of the leak detection system and to a degree of saturation equal to one ($S_r = 1$). This is not surprising: in this case, capillarity over the entire thickness of the leak detection system and Darcy's flow are identical.

This is why there is no possible flow beyond line CD of Figure 2.6-4. In other words, if the leakage rate through the top liner tends to be larger than the limit given by CD in Figure 2.6-4, the leak detection system becomes saturated. The pressure buildup in the leak detection system automatically keeps the leakage rate through the top liner at a value consistent with curve CD in Figure 2.6-4. As a result of pressure buildup in the leak detection system, leakage rate through the bottom liner and into the ground increases.

### 2.6.2.5 Conclusions

#### 2.6.2.5.1 Discussion of the Results

Although the bidimensional analytical study could be further refined, it has provided useful results which are summarized in Tables 2.6-1 through 2.6-5 and in Figures 2.6-2 through 2.6-5. These results are discussed below.

- Detection Time

Table 2.6-1 shows that a detection time smaller than one day can be achieved with a 2% slope and a minimum hydraulic conductivity of the leak detection system of $10^{-2}$ m/s (1 cm/s). It is reiterated that detection times are determined assuming steady-state conditions. Initial detection times, as discussed below, are much larger.

- Initial Detection Time

The results presented in Table 2.6-5 (established from Figure 2.6-4) show that initial detection times depend on leakage rate and are high. For example, if the leak detection system has a hydraulic conductivity of $10^{-2}$ m/s (1 cm/s) this study has shown that for bidimensional flow it will take 10 days for the initial detection of large uniform top liner leakage into an initially dry granular leak detection layer (leakage rate on the order of 1000 Ltd (1000 gpad)) and 1000 days for a leakage rate on the order of the proposed Action Leakage Rate (ALR) (10 Ltd (10 gpad)).

As shown in Table 2.6-6, initial leak detection times are four times shorter than the times given in Table 2.6-5 if the potential capillary zone of the leak detection system has an initial degree of saturation of 75% ($S_{ro}$ in Equation 2.6-39 and 40). Such an initial degree of saturation may result from: (i) water entrapped in the leak detection system during construction; (ii) water used to compact the low-permeability soil, if any, included in the top liner; (iii) ground water seeping into the leak detection system; and (iv) previous leakage.

- Detection Sensitivity

One of the assumptions of the analyses presented in Section 2.6.2 was that the bottom liner is perfectly impermeable. In steady-state conditions, this assumption leads to a zero detection sensitivity. To obtain a detection sensitivity different from zero, leakage through the bottom liner should be considered, which is done in the technical background document to EPA's April 17, 1987 Notice of Availability of Data on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments". This document shows that the detection sensitivity is smaller than 1 Ltd (gpad) if a composite bottom liner is used, and on the order of 100 Ltd (gpad) if a compacted soil bottom liner is used.

- Influence of Capillarity on Initial Detection Times

It appears from the charts (Figures 2.6-3 through 2.6-5) that capillarity plays a very important role and significantly delays initial detection. It is possible that the fact that the study is bidimensional tends to exaggerate the influence of capillarity (since in a three-dimensional study, concentrated leakage could be modeled, which is more realistic than uniform leakage). It is therefore possible that a three-dimensional study would show that the flow is governed more by Darcy's equation than by capillarity and therefore is faster. In other words, it is expected that initial detection times would be shorter in a three-dimensional study, especially for small leakage rates.

The influence of capillarity is further exaggerated in Table 2.6-5 and in the charts (Figures 2.6-3 through 2.6-5) because the leak detection system is assumed to be dry at the beginning of flow. Shorter initial detection times are obtained if a leak develops in a leak detection system which already contains liquid held by capillarity from various sources such as: previous leaks, construction water, or water expelled from the compacted soil component of a composite top liner. For example, initial leak detection times are divided by 4 if the initial degree of saturation of potential capillary zone in the leak detection system is 75% instead of 0%, as shown by Equations 2.6-39 and 40, and illustrated in Tables 2.6-5 and 2.6-6.

### 2.6.2.5.2 Extension of the Study

The study presented in Section 2.6.2 has provided useful results. Additional refinements and analyses to the study could be conducted to gain further insight into the performance of leak detection systems. These refinements and additional analyses are outlined below.

- Leakage through Bottom Liner

In the analyses presented in Section 2.6.2, the bottom liner is assumed to be absolutely impermeable. Similar analyses could be conducted with a range of permeabilities for the bottom liner to include cases such as: (i) composite bottom liner with a leaking FML; and (ii) low-permeability soil bottom liner (i.e., without a FML).

- Three-dimensional Study

The study presented in Section 2.6.2 is bidimensional. A similar study could be conducted considering concentrated leaks and using flow considerations (as governed by leak detection system material conductivity) to evaluate the width of the wetted area.

It is expected that such a three-dimensional study will indicate a lesser influence of capillarity than the bidimensional study presented in Section 2.6.2. If this is the case, smaller initial detection times would be obtained.

A major advantage of a three-dimensional study is the ability to consider a variety of leakage scenarios, including "worst cases" of concentrated leaks, while the bidimensional study presented in Section 2.6.2 considers a leakage equivalent to an average leakage rate per unit area, which leads to some semi-paradoxical results, such as initial detection times independent from distance between leak and sump.

Finally, experience gained in conducting a three-dimensional study including various leakage scenarios will make it possible to develop guidance for designers who have to face the difficult problem of selecting leakage scenarios.

- Parametric Study

Using the bidimensional and the three-dimensional studies, a parametric study should be conducted to produce results such as those given in Tables 2.6-1, 2.6-2, 2.6-5 and 2.6-6 and showing the influence of the following parameters:

- hydraulic conductivity of the leak detection system material;

- degree of saturation of the leak detection system material (resulting from previous leakage, construction water, water expelled from adjacent clay by consolidation);

- thickness and hydraulic transmissivity of the leak detection system;

- slope of the leak detection system;

- distance between collector pipes in a leak detection system;

- permeability of the bottom liner (which affects in particular the detection sensitivity); and

- leakage rate.

## 2.6.3 Two-Dimensional Numerical Study

### 2.6.3.1 Introduction

#### 2.6.3.1.1 Purpose of this Section

The purpose of this section is similar to the purpose of Section 2.6.2: it is to present an analysis of the functioning of leak detection systems which provides numerical information on key parameters involved in the proposed Liner/Leak Detection Rule. These parameters are:

- detection sensitivity;

- detection time; and

- Action Leakage Rate.

This study was performed by The Radian Corporation, Austin, Texas, and is described in [Radian, 1987a, b, c].

### 2.6.3.1.2 Approach

The approach chosen was to conduct a numerical study using a finite element computer program. Numerical methods have capabilities that analytical methods do not have. In particular, numerical studies can accept input in either analytical or numerical form, while, with analytical studies, input should be made in analytical form. Therefore, experimental data (such as those related to capillary retention) must be approximated by equations to be used in an analytical study, while they can be used without any approximation in numerical studies.

### 2.6.3.1.3 Complementarity of Analytical and Numerical Studies

One intent of the numerical study was to evaluate the influence of the bottom liner on the performance of the leak detection system. As a consequence, compacted soil liners as well as composite liners are considered for the bottom liner, and leakage through the bottom liner is taken into account. The analytical study was only intended to evaluate the performance of the drainage layer which constitutes the leak detection system and, consequently, leakage through the bottom liner is neglected. On the other hand, the analytical study was used to provide detailed information on steady-state flow as well as transient flow, while the numerical study concentrated mostly on transient flow from the beginning of leakage to the establishment of steady-state flow.

In the case of the analysis of the functioning of leak detection systems, because of the newness and the difficulty of the subject, it is appropriate that two studies conducted with two different approaches and by two different teams are available: the analytical study by GeoServices and the numerical study by Radian. Results can be compared and better conclusions can be drawn.

### 2.6.3.1.4 Organization of this Section

Section 2.6.3 is organized as follows:

- Section 2.6.3.2 presents the method used, i.e., the finite element computer program used for the numerical study, and the assumptions.

- Section 2.6.3.3 presents the results related to the initial detection time using the two-dimensional numerical study, and compares results of the numerical study with results of the analytical study.

## 2.6.3.2 Method

### 2.6.3.2.1 Description of the Finite Element Program

The finite element program used in the Radian study is UNSAT2D, a two-dimensional finite element computer program prepared by S.S. Papadopulos & Associates, Inc. to simulate soil moisture movement within waste disposal units including landfills, surface impoundments, and waste piles. Input parameters to the program include: (i) water movement across model boundaries and/or hydraulic head on model boundaries; (ii) land disposal unit geometry; (iii) material properties; and (iv) initial moisture conditions in the land disposal unit and surrounding soils. The program simulates the transient-state distribution of hydraulic head and soil moisture within the land disposal unit for each defined time step.

The program simulates a two-dimensional section through a land disposal unit. In the formulation of the program it has been assumed that adjacent parallel sections are identical in their physical and hydrological characteristics. As a result, the program can only model linear tears in the FML (holes cannot be modeled).

The program models FMLs and geotextiles as one-dimensional (linear) elements which have zero moisture storage. Leakage across the element is proportional to the head difference across the element. Soil and waste are modeled by two-dimensional, triangular elements which have moisture storage capacity.

2.6.3.2.2 Assumptions

- Leak Detection System Geometry

The considered leak detection system has the geometry shown in Figure 2.6-6. The key features are:

- 4H/1V (i.e., 25%) side slopes and 2% bottom slopes;
- 10 m (30 ft) long side slopes in horizontal projection and 18 m (60 ft) distance between collector pipes on the bottom; and
- 0.3 m (1 ft) thick leak detection layer.

- Leak Detection System Material

The pervious material used in the leak detection system has the following characteristics:

- porosity n = 0.3;
- a saturated hydraulic conductivity, $k_d$, of $10^{-5}$ m/s ($10^{-3}$ cm/s) in most cases, and $10^{-4}$ m/s ($10^{-2}$ cm/s) in some cases; and
- when unsaturated, the leak detection system material exhibits capillary suction given by suction-degree of saturation curves directly input into the computer program.

- Supply of Liquids into the Leak Detection System

At the beginning of each run of the computer program, the leak detection system is assumed to be approximately 75% saturated as a result of water entrapped during construction or water resulting from previous leakage. Then, the only supply of liquid to the leak detection system is leakage through the top liner.

UNSAT2D holds the leakage rate through the top liner constant and the hydraulic head on the bottom liner is allowed to vary. The leakage rate through the top liner was controlled by varying the hydraulic head on the top liner and the properties of the top liner material. Three types of top liner leaks were considered in the UNSAT2D numerical simulations (Figure 2.6-6b):

- uniform leakage through the entire top liner (uniform leak);
- leakage through a 3 m (10 ft) wide portion of the top liner on facility side slope (sidewall leak); and
- leakage through a 3 m (10 ft) wide portion of the top liner on the lower side (i.e., the bottom) of the land disposal unit (bottom leak).

A limitation of the program is that the smallest top liner sidewall and bottom leak which could be analyzed is 3 m (10 ft) wide. In reality, FML top liner field defects are more likely to be small tears or punctures, typically only a few millimeters (fraction of an inch) in diameter. Thus, the UNSAT2D top liner leak probably better represents leakage through a composite top liner than through a top liner consisting of an FML alone.

- Bottom Liner

A thin, very low-permeability layer with zero liquid storage capacity is placed over a compacted soil layer to simulate the FML component of a composite bottom liner. In the UNSAT2D simulations, the migration of liquid across this very thin layer is described by the FML leakance, which is defined as follows:

$$Q/A = L\ h \qquad \text{(Equation 2.6-48)}$$

where: Q = leakage rate; A = liner area; L = leakance; and h = hydraulic head. Recommended SI units are: Q ($m^3/s$); A ($m^2$); L ($s^{-1}$); and h = m.

It appears from Equation 2.6-48 that the leakance is identical to the classical permittivity, $\psi$.

In a double-liner system, the hydraulic head on top of the bottom liner is almost always very small. However, in the UNSAT2D simulations the capillary suction acting on the bottom of the FML component of the bottom liner is significant. In these simulations, this capillary suction is equivalent to a hydraulic head of 3.4 m (11.1 ft) acting on the FML from underneath. Such mechanism is unlikely to be effective in reality, and, to counteract the effect of the large hydraulic gradient set up in the numerical simulations by the action of capillary suction pulling water through the FML, a very low leakance value was selected by Radian. In UNSAT2D simulations, a leakance of $7 \times 10^{-14}$ $s^{-1}$ was selected to be used with a compacted soil capillary suction equivalent to 3.4 m (11.1 ft) of negative head. These values correspond almost exactly to a 1-mm (40-mil) thick FML with an equivalent hydraulic conductivity $k_g = 1 \times 10^{-14}$ m/s ($1 \times 10^{-12}$ cm/s) subjected to a hydraulic head of 30 mm (0.1 ft). Radian also carried out simulations with a FML leakance of $3 \times 10^{-11}$ $s^{-1}$. This leakance is 430 times larger than the one for an "intact" FML and can be considered to approximately represent the FML component of a bottom liner that has undergone "significant" deterioration. Finally, Radian carried out several numerical simulations with an intermediate leakance value, $L = 3 \times 10^{-12}$ $s^{-1}$, which may represent a FML that has undergone "some" deterioration.

- Flow Mechanism

The flow mechanism considered in the two-dimensional numerical study is identical to the flow mechanism considered in the two-dimensional analytical study:

- When leakage first occurs (i.e., when a first drop of liquid has just passed through the top liner), liquid is first held by capillarity in the leak detection system. The volume of liquid which can be held by capillarity depends on the opening size of the material used in the leak detection system and the moisture

- Some time after the beginning of leakage, enough liquid has passed through the top liner to saturate a sufficient volume in the vicinity of one of the collector pipes to initiate drain flow. This point is the beginning of the detection of leakage in the collector pipes and gives the value of the initial detection time defined in Section 2.6.1.3.3.

### 2.6.3.3 Results of the Numerical Study

#### 2.6.3.3.1 Summary of the Results

As indicated in Section 2.6.3.1.3, low-permeability soil bottom liners as well as composite bottom liners are considered in the numerical study. Only results related to composite bottom liners are considered here.

The numerical study shows that saturation generally occurs first in the vicinity of the collector pipe located at the center of the bottom of the land disposal unit (see Figure 2.6-6). Therefore, leakage is detected first at this pipe at a time that is the initial detection time considered here. However, some additional time is required for steady-state flow to be established. This time was also determined in the numerical study, but is not considered here.

As indicated in Section 2.6.3.2.2, three types of leaks are considered. Table 2.6-7 shows that, according to the numerical study there is no significant difference between the various types of leaks regarding initial detection times.

As indicated in Section 2.6.3.2.2, three cases are considered regarding the condition of FML component of the bottom liner: (i) intact FML; (ii) FML with significant deterioration; and (iii) an intermediate case. Table 2.6-7 shows that the condition of the bottom liner does not significantly affect the initial detection times for the considered cases. However, if smaller leakage rates had been considered, the effect of the condition of the bottom liner would have been more marked.

### 2.6.3.3.2 Comparison between the Analytical and Numerical Study

One of the results of the two-dimensional analytical study is Equation 2.6-40 which gives the leak detection time as a function of the leakage rate through the top liner. This equation was used with the following values of its parameters:

- an initial degree of saturation of the leak detection system of 75%, which is approximately the initial degree of saturation used in the two-dimensional numerical study; and

- a leak detection system material hydraulic conductivity of $10^{-4}$ m/s ($10^{-2}$ cm/s) although the hydraulic conductivity used in the numerical study was $10^{-5}$ m/s ($10^{-3}$ cm/s), because the analytical study has shown that, for a given top liner leakage rate, all hydraulic conductivities smaller than $10^{-4}$ m/s ($10^{-2}$ cm/s) lead to values of the leak detection time equal to the value calculated for $10^{-4}$ m/s ($10^{-2}$ cm/s), if the thickness of the leak detection system is 0.3 m (1 ft) or less.

Leak detection times were calculated using Equation 2.6-40 for the five values of the top liner leakage rate used in the two-dimensional numerical study. (These values are listed in the first line of Table 2.6-7.) The leak detection times thus calculated are given in the second line of Table 2.6-7.

Table 2.6-7 shows that there is a very good agreement between leak detection times calculated using the numerical method on one hand, and the analytical method on the other hand. This is remarkable since the leak detection system geometries considered in the two studies were significantly different, which appears by comparing Figures 2.6-1 and 2.6-6.

The good agreement between the results of the two studies confirms the validity of the analytical study. This is particularly interesting since the analytical study has been used for a variety of values of the hydraulic conductivity, and not only for $k_C = 10^{-5}$ m/s ($10^{-3}$ cm/s) like the numerical study. In fact, the recommended value for leak detection system hydraulic conductivity is $10^{-3}$ m/s (0.1 cm/s) for granular leak detection systems and $10^{-2}$ m/s (1 cm/s) for synthetic leak detection systems, as discussed in Section 2.8.

It may be concluded that the conclusions of the analytical study, presented in Section 2.6.2.5.2 and Tables 2.6-5 and 2.6-6, are confirmed by the numerical study and, therefore, can be used in subsequent sections. Further, the good agreement between the analytical and numerical studies helps to validate the extensive results of the numerical study presented in [Radian, 1987a, b, c].

Table 2.6-1. Leak detection time as a function of the distance, L, between the leak and the collector pipe or sump, and the hydraulic conductivity of the leak detection system material. This table has been established assuming steady-state flow in the leak detection system and an impermeable bottom liner. The slope of the leak detection system is 2%. The same results can be found in Figure 2.6-2. [Result of the GeoServices analytical study]

| | Hydraulic conductivity of the leak detection system material, $k_d$ | | | |
|---|---|---|---|---|
| L | $10^{-4}$ m/s ($10^{-2}$ cm/s | $10^{-3}$ m/s ($10^{-1}$ cm/s) | $10^{-2}$ m/s (1 cm/s) | $10^{-1}$ m/s (10 cm/s) |
| 15 m (50 ft) | 26 | 3 | 0.3 | 0.03 |
| 30 m (100 ft) | 52 | 5 | 0.5 | 0.05 |
| 50 m (165 ft) | 87 | 9 | 0.9 | 0.01 |
| 60 m (200 ft) | 104 | 11 | 1.1 | 0.1 |
| 100 m (330 ft) | 174 | 18 | 2 | 0.2 |
| | Leak detection times in days | | | |

Table 2.6-2. Required hydraulic transmissivity, $\theta_d$, for the leak detection system (in m²/s) as a function of leakage rate, q, and distance, L, between leak and collector pipe or sump for a leak detection system slope of 2% and an impermeable bottom liner. [Results from the GeoServices analytical study]

| | Top liner leakage rate per unit area, q liters/1000m²/day (gallons/acre/day) | | |
|---|---|---|---|
| L | 1000 | 10 000 | 100 000 |
| 15 m (50 ft) | $1.2 \times 10^{-5}$ | $1.2 \times 10^{-4}$ | $1.2 \times 10^{-3}$ |
| 30 m (100 ft) | $1.8 \times 10^{-5}$ | $1.8 \times 10^{-4}$ | $1.8 \times 10^{-3}$ |
| 50 m (165 ft) | $2.9 \times 10^{-5}$ | $2.9 \times 10^{-4}$ | $2.9 \times 10^{-3}$ |
| 60 m (200 ft) | $3.6 \times 10^{-5}$ | $3.6 \times 10^{-4}$ | $3.6 \times 10^{-3}$ |
| 100 m (330 ft) | $5.8 \times 10^{-5}$ | $5.8 \times 10^{-4}$ | $5.8 \times 10^{-3}$ |
| | Required hydraulic transmissivity for the leak detection system, $\theta_d$, in m²/s | | |

Table 2.6-3. Required thickness, D, for a leak detection system as a function of hydraulic conductivity, $k_d$, of the leak detection system material and the distance, L, between the leak and the collector pipe or sump. These results are related to a leakage rate of 10 000 liters/1000$m^2$/day (10,000 gallons/acre/day), a slope of 2% and an impermeable bottom liner. [Results from the GeoServices analytical study]

| | | Hydraulic conductivity, $k_d$ | | |
|---|---|---|---|---|
| Slope | L | 0.01 m/s<br>(1 cm/s) | 0.1 m/s<br>(10 cm/s) | 1 m/s<br>(100 cm/s) |
| 2% | 15 m<br>(50 ft) | 9 mm<br>(0.3 in.) | 0.9 mm<br>(0.03 in.) | 0.1 mm<br>(0.003 in.) |
| | 60 m<br>(200 ft) | 35 mm<br>(1.3 in.) | 3.5 mm<br>(0.13 in.) | 0.3 mm<br>(0.013 in.) |
| | 100 m<br>(330 ft) | 58 mm<br>(2.5 in.) | 5.8 mm<br>(0.25 in.) | 0.6 mm<br>(0.025 in.) |
| 4% | 15 m<br>(50 ft) | 4 mm<br>(0.17 in.) | 0.4 mm<br>(0.017 in.) | 0.04 mm<br>(0.0017 in.) |
| | 60 m<br>(200 ft) | 17 mm<br>(0.64 in.) | 1.7 mm<br>(0.064 in.) | 0.17 mm<br>(0.006 in.) |
| | 100 m<br>(330 ft) | 29 mm<br>(1.3 in.) | 2.9 mm<br>(0.13 in.) | 0.3 mm<br>(0.013 in.) |

Table 2.6-4. Values of capillary rise as a function of the hydraulic conductivity of the drainage medium used in the leak detection system.

| Hydraulic Conductivity $k_d$ | | Capillary Rise $h_c$ | | |
|---|---|---|---|---|
| (m/s) | (cm/s) | (m) | (mm) | (in.) |
| $1 \times 10^{-6}$ | $1 \times 10^{-4}$ | 3.10 | 3100 | 122 |
| $5 \times 10^{-6}$ | $5 \times 10^{-4}$ | 1.39 | 1386 | 54.6 |
| $1 \times 10^{-5}$ | $1 \times 10^{-3}$ | 0.98 | 980.3 | 38.6 |
| $5 \times 10^{-5}$ | $5 \times 10^{-3}$ | 0.44 | 438.4 | 17.3 |
| $1 \times 10^{-4}$ | $1 \times 10^{-2}$ | 0.31 | 310.0 | 12.2 |
| $5 \times 10^{-4}$ | $5 \times 10^{-2}$ | 0.14 | 138.6 | 5.5 |
| $1 \times 10^{-3}$ | $1 \times 10^{-1}$ | 0.10 | 98.0 | 3.9 |
| $5 \times 10^{-3}$ | $5 \times 10^{-1}$ | 0.044 | 43.8 | 1.7 |
| $1 \times 10^{-2}$ | 1 | 0.031 | 31.0 | 1.2 |
| $5 \times 10^{-2}$ | 5 | 0.014 | 13.9 | 0.55 |
| $1 \times 10^{-1}$ | 10 | 0.010 | 9.8 | 0.39 |
| $5 \times 10^{-1}$ | 50 | 0.004 | 4.4 | 0.17 |
| 1 | 100 | 0.003 | 3.1 | 0.12 |
| 5 | 500 | 0.0014 | 1.4 | 0.055 |

Table 2.6-5. Initial detection time (in days) as a function of the top liner leakage rate and the hydraulic conductivity of the leak detection system material for a 2% slope and an impermeable bottom liner. The leak detection system material is assumed to be initially dry. [Results from the two-dimensional analytical study]

| Hydraulic Conductivity of Leak Detection | Top liner leakage rate per unit area, q liters/1000m²/day (gallons/acre/day) | | | |
|---|---|---|---|---|
| $k_d$ | 1 | 10 | 100 | 1000 |
| Pea Gravel<br>$10^{-2}$ m/s<br>(1 cm/s) | 10000 | 1000 | 100 | 10 |
| Gravel<br>10 m/s<br>(10 cm/s) | 3000 | 300 | 30 | 3 |
| Coarse Gravel<br>1 m/s<br>(100 cm/s) | 1000 | 100 | 10 | 1 |

Initial detection times in days

Table 2.6-6. Initial detection time (in days) as a function of the top liner leakage rate and the hydraulic conductivity of the leak detection system material for a 2% slope and an impermeable bottom liner. The potential capillary zone is assumed to have a degree of saturation of 75%. [Results from the two-dimensional analytical study]

| Hydraulic Conductivity of Leak Detection $k_d$ | Top liner leakage rate per unit area, q liters/1000m$^2$/day (gallons/acre/day) | | | |
|---|---|---|---|---|
| | 1 | 10 | 100 | 1000 |
| Pea Gravel<br>$10^{-2}$ m/s<br>(1 cm/s) | 2500 | 250 | 25 | 2.5 |
| Gravel<br>$10^{-1}$ m/s<br>(10 cm/s) | 750 | 75 | 7.5 | 0.8 |
| Coarse Gravel<br>1 m/s<br>(100 cm/s) | 250 | 25 | 2.5 | 0.3 |
| | Initial detection times in days | | | |

Table 2.6-7. Comparison between the results of the two-dimensional analytical study (GeoServices study) and the two-dimensional numerical study (Radian study). Initial detection times ( in days) are given as a function of the top liner leakage rate. In both cases, the leak detection system hydraulic conductivity is smaller than $10^{-4}$ m/s ($10^{-2}$ cm/s) (the two-dimensional analytical study has shown that hydraulic conductivities, $k_c$, smaller than $10^{-4}$ m/s ($10^{-2}$ cm/s) lead to initial detection times identical to the initial detection times related to $k_c = 10^{-4}$ m/s ($10^{-2}$ cm/s)). The initial leak detection times of the analytical study have been obtained with an initial degree of saturation $S_{ro}$ = 75%, which is approximately identical to the degree of saturation at the beginning of leakage in the numerical study. The analytical study assumes an ideally impermeable bottom liner and should therefore be compared to the first of the three cases considered in the numerical study. (Note: only the numerical study cases with a composite bottom liner are considered in this table.) Legend: (u) = uniform leak; (s) = sidewall leak (see Figure 2.6-6b.)

| | | Leakage rate per unit area through the top liner, q, gallons/acre/day | | | | |
|---|---|---|---|---|---|---|
| | | 49 | 60 | 92 | 780 | 1238 |
| GeoServices (analytical) | | 520 | 425 | 277 | 33 | 21 |
| Radian (numerical) | intact bottom FML | 600<br>(s) | 495<br>(s) | --- | 30<br>(u) | --- |
| | intermediate case | --- | --- | 334<br>(u) | --- | 35<br>(u) |
| | deteriorated bottom FML | --- | 550<br>(s) | 351<br>(u) | 30<br>(u) | --- |
| | | Initial leak detection time, days | | | | |

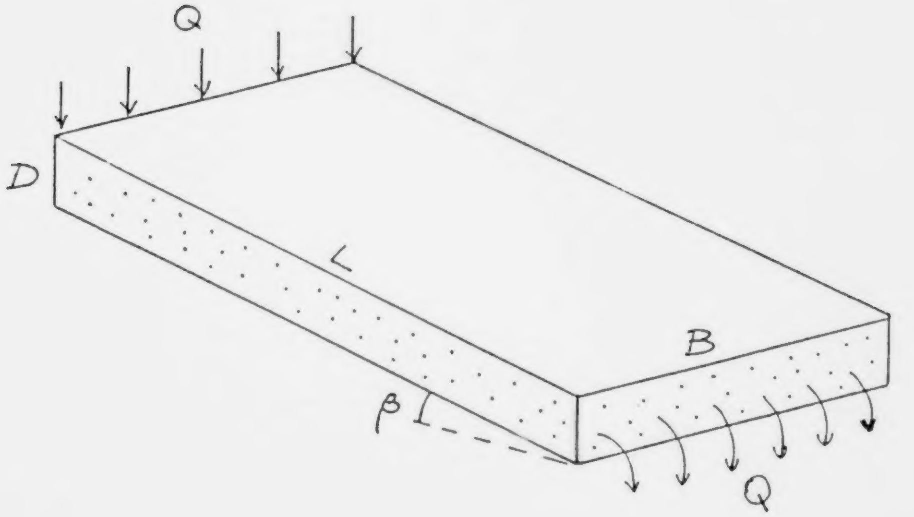


Figure 2.6-1. Geometry of the leak detection system considered in the two-dimensional analytical study.

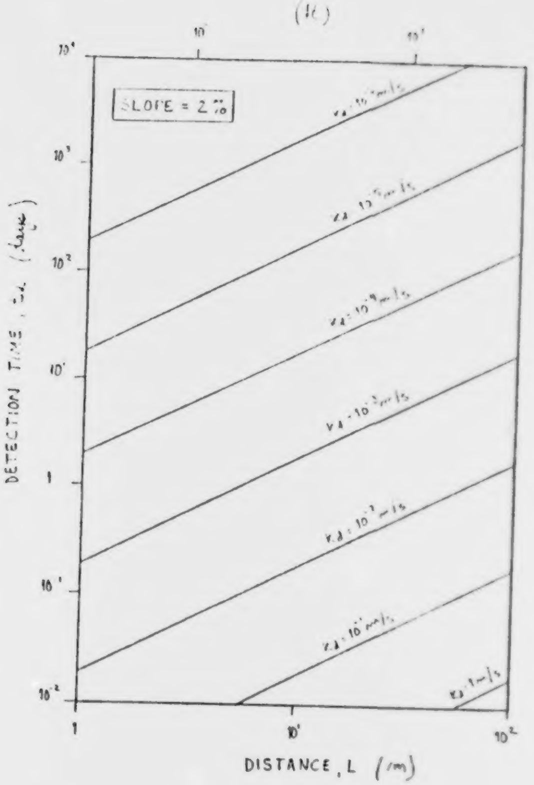


Figure 2.6-2. Leak detection time as a function of the distance between the leak and the collector pipe or sump, and the hydraulic conductivity of the leak detection system material. This chart has been established assuming steady-state flow in the leak detection system and an ideally impermeable bottom liner. The slope of the leak detection system is 2%. The same results can also be found in Table 2.6-1. [Results from the two-dimensional analytical study]

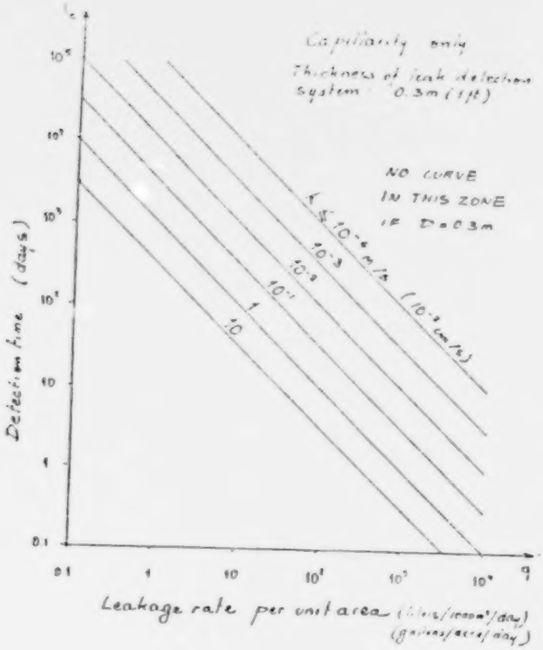


Figure 2.6-3. Leak detection time as a function of top liner leakage rate and hydraulic conductivity of the material used in the leak detection system [two-dimensional analytical study] assuming an ideally impermeable bottom liner and an initially dry leak detection system. The curves are independent of the thickness of the leak detection system provided that the thickness is greater than capillary rise (see Table 2.6-4). However, the fact that the last curve is valid for all hydraulic conductivities smaller than $10^{-4}$ m/s is due to the fact that the capillary rise for a granular material with a hydraulic conductivity of $10^{-4}$ m/s is 0.3 m, which is the thickness of the considered leak detection system. Caution: as explained in Figure 2.6-4, portions of the above curves are not applicable because they are superseded by curves related to Darcy's flow.

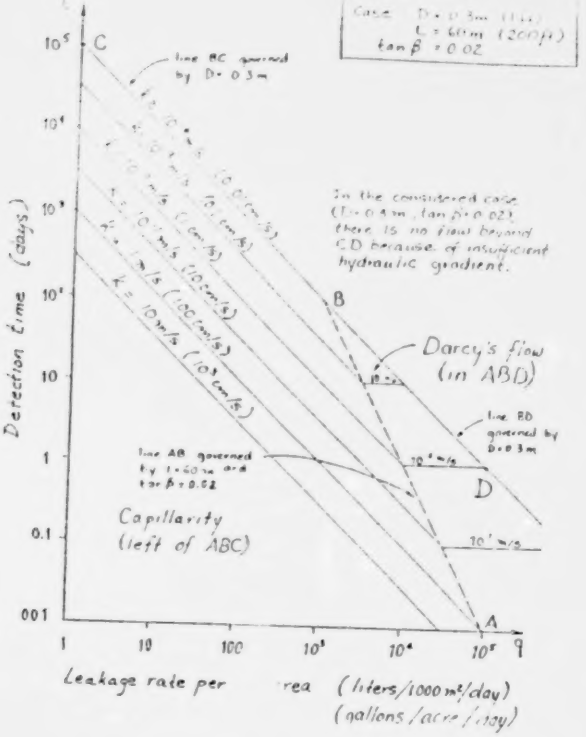


Figure 2.6-4. Detection time as a function of top liner leakage rate and hydraulic conductivity of the material used in the leak detection system [two-dimensional analytical study] assuming an ideally impermeable bottom liner and an initially dry leak detection system. The capillarity curves (lines at 45° left of ABC) are independent of the thickness D, the length L, and the slope β of the leak detection system. The Darcy's curves (horizontal lines, in zone ABD) are related to L = 60m (200 ft) and are independent of the thickness D. On a given Darcy's curve such as XY, the head on the bottom liner is zero in X and increases linearly between X and Y where it reaches its maximum value, that is, the thickness D of the leak detection system. The head on the bottom liner related to a point such as Y can even be greater than the thickness of the leak detection system if there is a large hole in the top liner and if the leakage through that hole is limited by hydraulic transmissivity of the leak detection system (thereby building up pressure in the leak detection system).

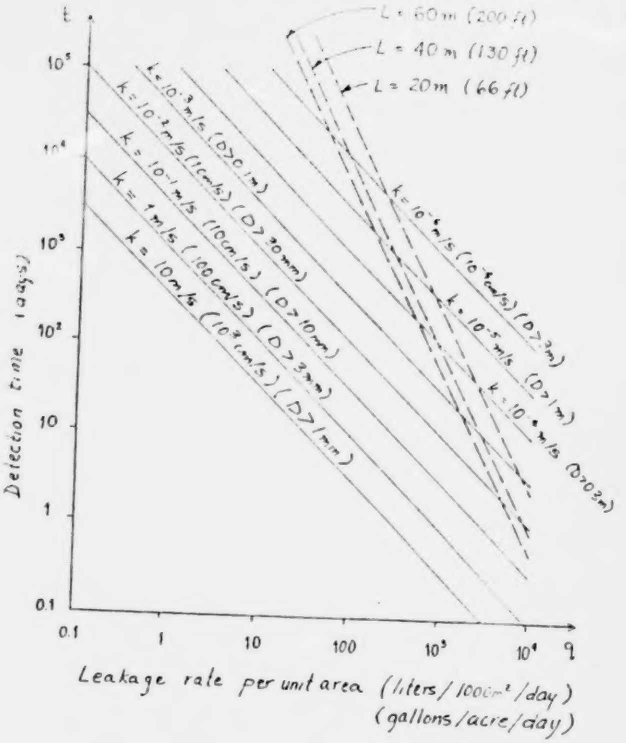


Figure 2.6-5. Detection time as a function of top liner leakage rate, hydraulic conductivity, and length of flow path [two-dimensional analytical study] assuming an impermeable bottom liner. Generic chart which can be used directly by expert user or can be used to generate series of charts such as those in Figure 2.6-5. The curves at 45° are related to capillarity. These curves are valid only if the thickness of the leak detection layer is larger than the value indicated on each curve. The curves at 67.5° are related to Darcy's flow.

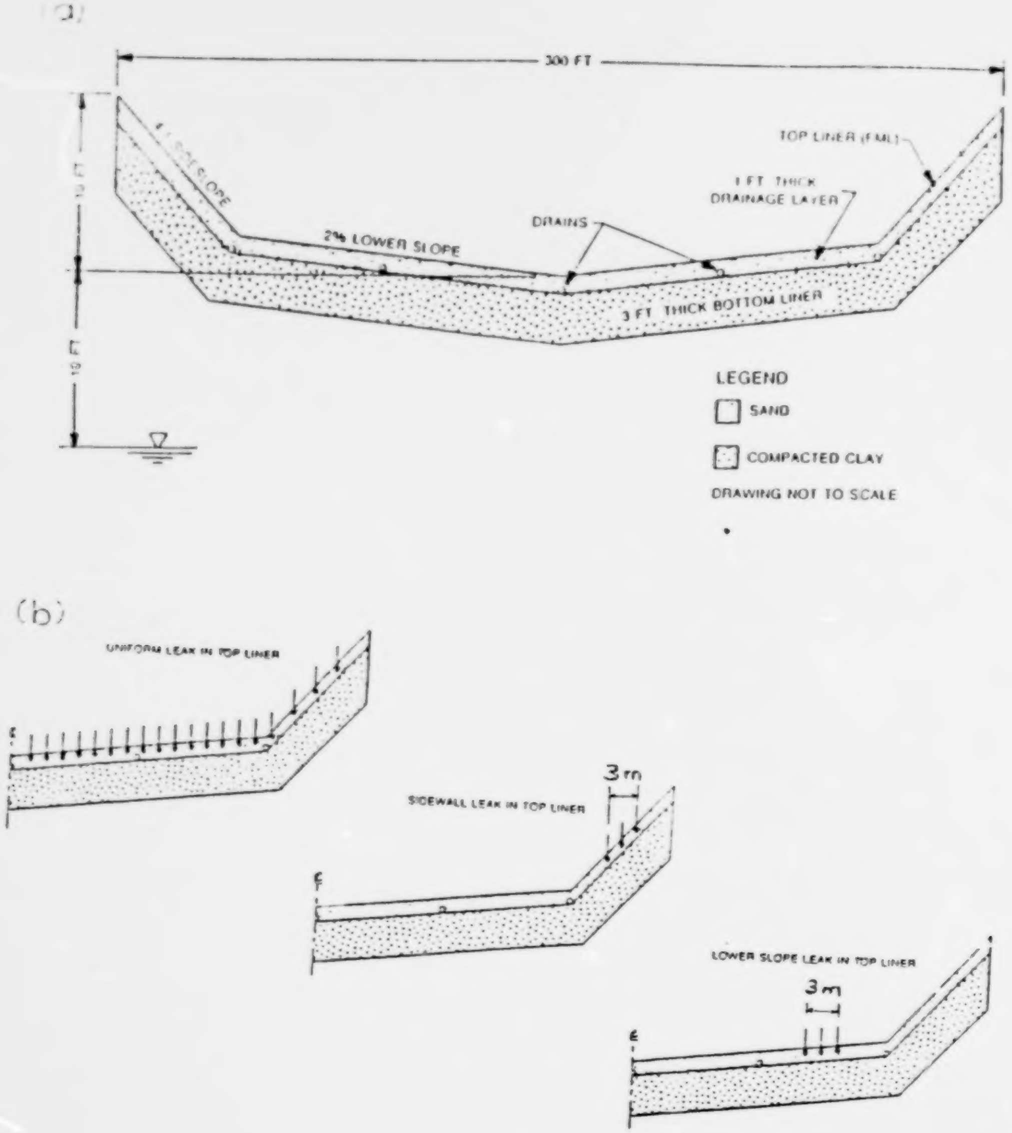


Figure 2.6-6. Lining system modeled using the UNSAT2D program, to study leak detection system performance: (a) geometry of the system; (b) definition of the three types of leaks considered [Radian, 1987a, b, c].

## 2.7 PERFORMANCE CRITERIA FOR LEAK DETECTION SYSTEMS

### 2.7.1 Introduction

### 2.7.1.1 Scope of the Section

#### 2.7.1.1.1 Purpose of the Section

The purpose of this section is to review the technical information pertinent to performance criteria for leak detection systems. This information supports:

- the selection of the leak detection system performance characteristics that are most appropriate to establish performance criteria; and

- the establishment of the criteria themselves, which are expressed as values (maximum or minimum, depending on the considered characteristics) that must be met by the selected performance characteristics.

#### 2.7.1.1.2 Organization of the Section

The introduction:

- defines the concept of "performance criteria" and related concepts; and

- reviews leak detection system performance characteristics that can be considered to establish performance criteria, and selects those that are deemed appropriate.

Then, the technical information relevant to the various performance criteria and the rationale for the establishment of these criteria are presented in Sections 2.7.2 through 2.7.4.

### 2.7.1.2 Design Requirements

The term "design requirements" and "leak detection capability" encompasses "performance criteria" and "design specifications". Both concepts are discussed below.

#### 2.7.1.2.1 The Concept of Performance Criteria

- Definition

Performance criteria quantify the minimum performance for which a leak detection system should be designed. In other words, the performance criteria considered here are design performance criteria, rather than field performance criteria.

- Demonstration

As a result of the above definition, the owner or operator of the hazardous waste management unit need not conduct a field test or to make field measurements to demonstrate that the leak detection system, when built, will meet the performance criteria. The owner or operator should only be required to demonstrate that the design is such that the leak detection system is expected to meet the performance criteria, provided that it is properly constructed.

#### 2.7.1.2.2 The Concept of Design Specifications

- Definition

Design specifications are requirements that selected physical characteristics of the leak detection system should satisfy, such as size of the system and properties of the materials used to construct the system.

- Demonstration

Demonstration by the owner that a leak detection system meets design specifications can be done in two stages:

- at the design stage, the owner or operator should select leak detection system geometry and materials that meet the design specifications; and

- at the construction stage, conformance testing of the materials and measurements should be made to verify that the materials used and the geometry of the leak detection system meet the design specifications and are in agreement with the design.

### 2.7.1.2.3 Performance Criteria and Design Specifications

– Differences Between Performance Criteria and Design Specifications

Performance criteria and design specifications are different:

- Performance criteria are related to the way a leak detection system must work as predicted by design.

- Design specifications are related to the way a leak detection system must be built.

The combination of performance criteria and design specifications is EPA's way of ensuring adequate leak detection capability in the LDCRS of double liner systems. It is believed to be wise to require a leak detection system to meet both performance criteria and design specifications because it appears that in many cases performance criteria will not be met unless design specifications are also imposed (which implies that it is important for the design specifications to be carefully chosen).

Design specifications appear to be essential. However, it should not be concluded that design specifications alone are sufficient and can replace a good design. Design specifications alone are not sufficient because, even if a leak detection system meets all design specifications, there is no guarantee that its field performance, or even its design performance, will meet the performance criteria.

- Complementarity of Performance Criteria and Design Specifications

One may argue that a leak detection system should meet its objectives regardless of the means involved. In other words, one may argue that only performance criteria are necessary and useful to ensure adequate leak detection capability while design specifications are unnecessary and useless. One may even argue that design specifications are detrimental because, by specifying the means to be used, they restrict innovation and prevent designers from developing and using new technologies. There is some merit to this second argument, and it should be understood that design specifications can be modified and adapted when knowledge increases and new technologies become available. The RCRA amendments of 1984, in fact, call for minimum technological requirements to be revised "from time to time to take into account improvements in the technology of control and measurement" RCRA 3004(o)(1). The recent history of the development of the use of synthetic drainage layers reflects EPA's willingness to adapt design specifications when a new technology becomes available which appears to be able to meet design objectives and performance criteria.

Design specifications are very useful, however, as shown by the following five-point discussion:

- Special expertise is required to design a leak detection system and to demonstrate that it will meet the performance criteria and provide adequate leak detection capability.

- Special expertise is also required to evaluate such designs and demonstrations.

- Because of the increasing use of synthetic drainage layers and the possible growing use of other innovative technologies, some designers, reviewers, and permit writers will lack the necessary expertise to competently evaluate and apply the new technologies, at least initially.

- Guidelines on leak detection system design are therefore needed by designers, reviewers, and permit writers.

- The simplest guidelines are design specifications. They are simple to follow and their implementation is simple to verify.

It may be concluded that design specifications are necessary safeguards against errors in design that otherwise would be undetected.

## 2.7.1.3 Performance Characteristics

### 2.7.1.3.1 Scope of the Section

- Purpose of this Section

The purpose of this section is: (i) to identify the performance characteristics, i.e., the parameters which characterize the performance of a leak detection system; and (ii) to select the characteristics that will be used to establish performance criteria.

- Organization of the Section

This section includes two subsections. The first subsection is devoted to a review of the leak detection capabilities that leak detection systems should have in general. In the second subsection, the capabilities of the leak detection system are expressed in terms of specific performance characteristics, relevant to the types of leak detection systems considered in this background document.

### 2.7.1.3.2 Leak Detection System Capabilities

- Purpose of Leakage Monitoring

There are two reasons for monitoring the presence of leakage between the two liners: (i) to prevent its causes; and (ii) to minimize its consequences. Causes and consequences are discussed hereafter.

Leakage is caused by leaks. To prevent the causes of leakage, leaks must be detected, located, and repaired. Leak location determination, therefore, is a required capability of a leakage monitoring system intended to provide information in order to prevent the causes of leakage.

The presence of leakage between the top and bottom liners of a double liner system has a detrimental consequence: a liquid head is created on the bottom liner. This liquid head will cause leakage into the bottom liner. If the bottom liner is defective and if the liquid head becomes large, leakage may pass through the bottom liner and into the ground. With EPA's "systems approach", the prevention of migration of hazardous constituents into the bottom liner and out of the unit can be minimized by proper design, construction and operation of two lining system components: the bottom liner and the leak detection system. With respect to the bottom liner, EPA has recently shown [USEPA, 1987] that composite bottom liners will minimize the potential for migration of hazardous constituents into the bottom liner. With respect to the leak detection system, the potential for migration into the bottom liner can be minimized by minimizing the hydraulic head acting on top of the bottom liner. Ideally, to ensure a minimum hydraulic head, the liquid head on the bottom liner should be measured at many locations. This is extremely difficult because of the amount of monitoring equipment that would have to be installed, and the complexity and cost of equipment maintenance and data collection. It is much more practical to make an indirect evaluation of liquid head by measuring leakage rates, since liquid head on the bottom liner is linked to the leakage rate through the top liner. Therefore, a leakage monitoring system, whose purpose is to provide useful information so that the consequences of the presence of leakage can be minimized, must have the capability of evaluating leakage rate.

Whether the purpose of monitoring leakage is to provide information for preventing leakage causes or minimizing leakage consequences, the rapidity with which the information is obtained is important.

In addition, a leak detection system must perform throughout the active life and post-closure care period of the land disposal unit. Therefore, the materials used to construct a leak detection system must have adequate durability when placed in contact with the leachate that results from the treatment, storage or disposal of hazardous wastes at a specific unit. This important performance requirement is discussed in Section 2.4.3.5.

#### - Identification of Leak Detection Systems Capabilities

The conclusion of the above discussion is that, for leak detection systems in general, the following capabilities can be considered:

- leak location determination, if the purpose is to prevent leakage causes, i.e., to repair leaks;
- leakage rate evaluation, if the purpose is to minimize leakage consequences; and
- rapidity with which the information is obtained, in both cases.

These general capabilities are discussed below to establish a list of performance characteristics relevant to the leak detection systems considered in this background document.

### 2.7.1.3.3 Selection of Performance Characteristics

#### - Purpose of the Section

The purpose of this section is to select, after considering available technologies, the performance characteristics of a leak detection system which should be used to establish performance criteria.

According to Section 2.7.1.3.2, the following leak detection system capabilities should be addressed in order to select relevant performance characteristics:

- leak location determination;
- evaluation of leakage rate; and
- rapidity of providing information.

These should be translated into simple performance characteristics which can be used to express performance criteria in a practical manner. This is the purpose of the following discussion.

- Discussion of Leak Location Determination

As discussed in Section 2.3.1, it is very difficult and expensive to identify the location of leaks. In addition, there is not much incentive for finding the location of leaks:

- In landfills, the occurrence of significant rates of leakage are unlikely after closure of the unit. If major leakage through the top liner is detected during landfill operation, a typical remedial action consists of capping a certain area of the unit (i.e., covering the waste with a liner) to prevent infiltration of rainwater, thereby drastically reducing leachate production. It is not necessary to know exactly where leakage occurs to design the capping of an area of a landfill. It is, however, useful to subdivide a large landfill into cells with independent leak detection systems to be able to identify in which area leakage occurs.

- In surface impoundments, the strategy in case of major leakage is different. The impoundment can be emptied and the liner repaired or replaced ("retrofitted"). It is therefore useful to know the location of leaks. However, leaks can be located by means that do not need to be built in the lining system. These means include portable electric resistivity equipment, vacuum testing, and visual inspection of the lining system after emptying the impoundment.

- In waste piles, leak location determination is usually not needed because of the small size and the temporary nature of these units.

It therefore appears that, from a technical standpoint, it is not crucial to build into the leak detection system a permanent means to identify leak location (although owners or operators may elect to do this voluntarily). This conclusion is corroborated by the discussion in Section 2.3.2 that led to the recommendation to use, as a leak detection system, the leachate collection and removal system located between the two liners, which cannot determine leak location.

Although the ability to determine the location of leaks is a desirable characteristic of a leak detection system, it appears from the above discussion that it is neither necessary nor practical (with presently available technology) to consider the determination of leak location as a performance characteristic for which a general performance criterion should be established. However, if a leak detection system, which meets all general performance criteria and design specifications, also has the capability of locating leaks, it clearly has an additional benefit.

- Discussion of Leakage Rate Evaluation

The ability of a leak detection system to evaluate leakage rates can be expressed by two performance characteristics:

- the leachate collection efficiency; and
- the leak detection sensitivity.

These two performance characteristics can be defined as follows:

- The leachate collection efficiency is the ratio between leakage rate conveyed by the leak detection system and collected at the sump, and the leakage rate that impinges the leak detection system (i.e., the leakage rate that has passed through the top liner).

- The leak detection sensitivity is the smallest leakage rate that can be detected.

As will be discussed in Section 2.7.4, it is impractical (although it is desirable) to establish a leak detection system performance criterion on the basis of the leachate collection efficiency. Therefore the only performance characteristic related to leakage rate evaluation that will be considered in the establishment of performance criteria is the leak detection sensitivity (which will be discussed in detail in Section 2.7.2).

- Discussion of Speed at which Information is Provided

The speed at which information is provided by the leak detection system can be expressed by the "detection time" which is essentially the time necessary to detect leakage which originates in a leak as far away as possible from the sump. The concept of detection time is discussed in detail in Section 2.7.3.

- Recapitulation of Performance Characteristics

As a result of the above discussion, the following performance characteristics are considered:

- detection sensitivity; and
- detection time.

These two performance characteristics are discussed in detail in Sections 2.7.2 and 2.7.3.

## 2.7.2 Detection Sensitivity

### 2.7.2.1 Introduction

### 2.7.2.1.1 Scope of the Section

- Purpose of the Section

The purpose of this section is: (i) to present the technical information available to support the establishment of a performance criterion regarding detection sensitivity; and (ii) to establish the detection sensitivity performance criterion.

- Organization of the Section

The introduction includes a definition of detection sensitivity and a discussion of the importance of this concept. A review of the available information and a rationale for establishing the performance criterion is presented in Section 2.7.2.2. The criterion is presented in Section 2.7.2.3, with a discussion.

### 2.7.2.1.2 Definition and Importance of Detection Sensitivity

- Definition

The detection sensitivity of a leak detection system can be defined as:

> Detection sensitivity is the smallest quantity of liquid that can pass through the top liner and be detected by the leak detection system.

- Importance of Detection Sensitivity

Detection sensitivity is an essential performance characteristic of any leakage monitoring system, as indicated in Section 2.7.1.3.3. In addition, a high leak detection sensitivity is consistent with the statutory requirements of the RCRA amendments which require the leak detection system be able to detect leakage "at the earliest practicable time".

A leak detection system with a high sensitivity (i.e, able to detect a small leakage rate) permits a detailed monitoring of the behavior of a land disposal unit. This is very useful to the owner or operator of the considered unit and it is useful in general, in that it increases knowledge about the functioning of lining systems used in land disposal units. The experience thus gained will benefit the designers, the owners, the operators, and the environment.

### 2.7.2.2 Establishment of the Detection Sensitivity Criterion

#### 2.7.2.2.1 Summary of Relevant Information

Available technical information which can be used to establish the detection sensitivity performance criterion can be summarized as follows:

- Table 2.7-1 gives calculated leakage rates through FML and composite top liners. This table (which is a reproduction of Table 2.2-16) shows that leakage rate under a hydraulic head of 0.3 m (1 ft), which is the specified maximum permissible head on the top liner of a landfill, is on the order of 0.3 Ltd (gpad). This table also shows that leakage rate under a hydraulic head of 0.03 m (0.1 ft), which may represent a typical average head on the top liner of a landfill, is on the order of 0.02 Ltd (gpad).

- Table 2.7-1 also shows that leakage rate under a hydraulic head of 0.03 m (0.1 ft), which is conservative for a bottom liner under almost all operating conditions, is very small: 0.001 Ltd (gpad) due to permeation through the FML, and 0.02 Ltd (gpad) due to one standard hole or defect in the FML component of a composite liner (i.e., a 1 $\text{cm}^2$ (0.16 $\text{in}^2$.) hole) per 4000 $\text{m}^2$ (acre) with good contact between the FML and compacted soil.

- Using Darcy's equation, it is possible to make a calculation showing that a bottom liner made exclusively of compacted soil (i.e., without a FML) would exhibit the following leakage rates (almost regardless of its thickness): approximately 900 Ltd (gpad) if the hydraulic conductivity of the compacted soil is $10^{-8}$ m/s ($10^{-6}$ cm/s), and approximately 90 Ltd (gpad) if the hydraulic conductivity of the compacted soil is $10^{-9}$ m/s ($10^{-7}$ cm/s).

The above technical information will be used in the next section.

### 2.7.2.2.2 Rationale for the Criterion

- Lower Boundary for the Criterion

Clearly, a lower boundary for the detection sensitivity is the leakage rate through the bottom liner. If a compacted soil bottom liner is considered, the leakage rate through the bottom liner (on the order of 90-900 Ltd (gpad) as given in the previous Section) will in most cases exceed the leakage rate through the top liner given in Table 2.2-16. As a result, no leakage will be detected. Therefore, a detection sensitivity of 90-900 Ltd (gpad) is too large to meet the requirement to detect leaks at the earliest practicable time.

If a composite bottom liner is used, the leakage rate will be 0.001 Ltd (gpad) as a result of permeation through the FML, and in the range of 0.02 Ltd (gpad) as a result of holes, as shown in Table 2.7-1. However, in evaluating detection sensitivity it is important to realize that holes should not be considered since there will always be some leakage that will bypass the holes and be detected. Therefore, when a composite bottom liner is used, a lower boundary for the detection sensitivity is 0.001 Ltd (gpad), which is the leakage rate due to permeation through the FML.

- Upper Boundary for the Criterion

A leak detection system should be capable of detecting leakage due to a standard FML hole. Then, if a hole develops in a FML during the active period or post-closure care period of a hazardous land disposal unit, leakage through the hole can be detected. Since both FML and composite top liners are allowed by EPA, both should be considered in setting the upper boundary of the detection sensitivity criterion. Since the composite liner will allow less top liner leakage than the FML, it will be more critical in setting the criterion. Therefore, according to Table 2.7-1, the smallest leakage rates due to holes in composite liners are:

- 0.02 Ltd (gpad) with a hydraulic head of 0.03 m (0.1 ft) which may be assumed to correspond to average conditions on the top liner of a landfill.

- 0.3 Ltd (gpad) with a hydraulic head of 0.3 m (1 ft) that is typically considered in the design of a landfill top liner.

- Conclusion

The lower boundary of 0.001 Ltd (gpad) is very small and would require a leakage rate measurement precision that is not typically feasible in the field. Therefore, a value close to the upper boundary should be considered, such as 0.1 Ltd (gpad). This rate of top liner leakage represents a very high level of detection sensitivity and can be considered to represent BDAT for leak detection systems.

### 2.7.2.3 Presentation of the Detection Sensitivity Criterion

#### 2.7.2.3.1 Expression of the Criterion

The proposed detection sensitivity performance criterion resulting from the above analysis would be as follows:

> The smallest leakage rate that can be detected by (the "detection sensitivity" of) a leak detection system should be 0.1 liter/1000m$^2$/day (0.1 gallon/acre/day).

This proposed value represents the best demonstrated available technology (BDAT) to the best of our knowledge. However, since the collection of liquid quantities as small such as 0.1 liter/1000m$^2$/day (0.1 gallon/acre/day) may not be possible in the field (due to the finite capabilities of collector systems, sumps, and pumps), a higher value of detection sensitivity, such as 1 liter/1000m$^2$/day (1 gallon/acre/day) should be considered.

### 2.7.2.3.2 Discussion

The above detection sensitivity performance criterion can be met only if the leak detection system is properly designed and constructed.

In order to design a leak detection system in a way that increases the detection sensitivity (i.e., that decreases the size of the smallest leakage rate that can be detected), the designer should recognize that it is necessary to decrease capillarity in order to minimize the amount of liquid held in the system. The following can be done to decrease capillarity:

- The leak detection system material should have a high hydraulic conductivity such as 10$^{-2}$ m/s (1 cm/s) to have a small capillary effect as indicated in Section 2.8.
- A judicious arrangement of saw-tooth slopes at the bottom of the landfill may concentrate the flow in channels, which favors Darcy's flow and reduces the relative importance of capillarity.

To properly construct a leak detection system, at least the following should be done:

- Drainage materials such as coarse sand, gravel or geonets should be clean. Therefore, they should be washed at the site, prior to installation (even if they look clean, because experience shows that they are never really clean), and they must be protected from dust and dirt prior to being covered by other materials of the lining system.

- FMLs used as the upper component of a composite bottom liner must be placed without significant wrinkles, and the quality assurance monitors should verify that the pattern of small wrinkles, which cannot be eliminated, do not form closed areas where liquids will pond. To that end, observations after a rainfall are very useful, especially if photographs are taken.

## 2.7.3 Detection Time

### 2.7.3.1 Introduction

### 2.7.3.1.1 Scope of the Section

- Purpose of the Section

The purpose of this section is: (i) to present the technical information available to support the establishment of a performance criterion regarding detection time; and (ii) to establish the detection time performance criterion.

- Organization of the Section

The introduction of this section includes a definition of detection time and a discussion of the importance of this concept. A review of the available information and a rationale for establishing the performance criterion is presented in Section 2.7.3.2. Finally, the criterion is presented in Section 2.7.3.3, with a discussion.

### 2.7.3.1.2 Definition and Importance of Detection Time

- Definition

The detection time of a leak detection system can be defined as:

> Detection time is the time from when liquid enters the leachate collection and removal system between the liners (i.e., the leak detection system) to when it is present (and available for removal) in the leachate collection and removal system sump.

- Comments

The proposed Liner/Leak Detection Rule will require a leak detection system that is capable of detecting leakage that migrates through the top liner into the space between the liners at the "earliest practicable time". This implies that the detection time should be short but not so short that it would be impractical to meet with currently available technology. The proposed performance criterion should take this consideration into account.

If a FML/compacted soil composite liner is used, is there leakage when leachate passes through the FML (which is the upper component of the composite liner) or when leachate passes through the soil (which is the lower component of the composite liner)? This issue has been debated in the past. But, clearly, there is one logical answer: since a composite liner is one liner, not two, there is leakage only when liquid has passed through both components. In fact, the definition given above is clear: the initial time for measuring the detection time is the time when leakage enters the leak detection system.

- Importance of Detection Time

Detection time is an essential performance characteristic of any leakage monitoring system as indicated in Section 2.7.1.3.3. In addition, strict detection time performance criteria are in agreement with the proposed Liner/Leak Detection Rule requirement to ensure detection "at the earliest practicable time".

A short detection time is important, especially for large leaks, because it is a prerequisite for prompt triggering of appropriate Response Action Plan (see Section 2.10).

## 2.7.3.2 Establishment of the Detection Time Criterion

### 2.7.3.2.1 Summary of Relevant Information

Available technical information which can be used to establish the detection time criterion is summarized in Table 2.7-2 which gives detection time as a function of the hydraulic conductivity of the leak detection system material and the distance between the leak and the collector pipe or the sump. This table is a reproduction of Table 2.6-1.

### 2.7.3.2.2 Rationale for the Criterion

- Presentation of the Rationale

The leak detection time should be shorter than the monitoring period so that any leakage event happening in a given monitoring period can be detected during that period. Therefore, if a daily monitoring of the leak detection sumps is required, a leak detection time of one day maximum is recommended.

As shown in Table 2.7-2, such a leak detection time would be achieved with a hydraulic conductivity of the leak detection system material of $10^{-2}$ m/s (1 cm/s) and a distance between the leak and the collector pipe or sump less than 50 m (165 ft).

## 2.7.3.3 Presentation of the Detection Time Criterion

### 2.7.3.3.1 Expression of the Criterion

As a result of the above analysis, the proposed detection time performance criterion of a leak detection system would be as follows:

> The time from when liquid enters the leachate collection and removal system between the liners (i.e., the leak detection system) to when it is available for removal in the leakage detection, removal, and collection sump should be smaller than one day assuming steady-state flow conditions.

The proposed value represents the best demonstrated available technology (BDAT) to the best of our knowledge. However, it can be replaced by other values for practical reasons.

#### 2.7.3.3.2 Discussion

The above detection time performance criterion can be met only if the leak detection system is properly designed and constructed.

In order to design a leak detection system in a way that decreases the detection time, the designer should recognize that it is necessary to decrease capillarity in order to minimize the amount of liquid held in the system. The following can be done to decrease capillarity:

- The leak detection system material should have a high hydraulic conductivity such as $10^{-2}$ m/s (1 cm/s) to have a small capillary effect as indicated in Section 2.8.
- A judicious arrangement of saw-tooth slopes at the bottom of the landfill may concentrate the flow in channels, which favors Darcy's flow and reduces the relative importance of capillarity.

To properly construct a leak detection system, at least the following should be done:

- Drainage materials such as coarse sand, gravel or geonets should be clean. Therefore, they must be washed at the site, prior to installation (even if they look clean, because experience shows that they are never really clean), and they must be protected from dust and dirt prior to being covered by other materials of the lining system.

- FMLs used as the upper component of a composite bottom liner must be placed without significant wrinkles, and the quality assurance monitors should verify that the pattern of small wrinkles, which cannot be eliminated, do not form closed areas where liquids will pond. To that end, observations after a rainfall are very useful, especially if photographs are taken.

## 2.7.4 Leachate Collection Efficiency

### 2.7.4.1 Introduction

#### 2.7.4.1.1 Scope of the Section

- Purpose of the Section

The purpose of this section is to discuss the concept of leachate collection efficiency and evaluate if it can be used as a performance criterion for a leak detection system.

- Organization of the Section

The introduction of this section includes a definition of leachate collection efficiency. A discussion of the concept is presented in Section 2.7.4.2.

#### 2.7.4.1.2 Definition of Leachate Collection Efficiency

The leachate collection efficiency of a leak detection system can be defined as:

> The leachate collection efficiency is the ratio of the leakage collected at the leak detection system sump divided by the leakage entering the leak detection system.

### 2.7.4.2 Discussion of the Concept of Leachate Collection Efficiency

#### 2.7.4.2.1 Analysis of Leakage Types

Figure 2.7-1 (which is a reproduction of Figure 2.1-3), illustrates that four leakages can be defined for a lining system:

- C = leakage from the LCRS into the top liner;
- E = leakage through the top liner (i.e., leakage which impinges into the leak detection system);
- H = leakage from the leak detection system into the bottom liner; and
- J = leakage through the bottom liner (i.e., leakage into the ground).

Clearly, leakage of hazardous leachate into the ground (leakage J in Figure 2.7-1) must be absolutely minimized - ideally, prevented. The following relationships exist between the four leakages:

- Leakage into the ground (J in Figure 2.7-1) is smaller than leakage from the leak detection system into the bottom liner (H in Figure 2.7-1) because a fraction of leakage H is absorbed by, or otherwise entrapped in, the bottom liner.
- Leakage H, from the leak detection system, is much smaller than leakage E, through the top liner, because a portion F of leakage E remains entrapped in the leak detection system (by capillarity or otherwise) and a portion G of leakage E is conveyed by the leak detection system to a sump where it is collected and from where it is removed.
- Leakage E, through the top liner, is smaller than leakage C into the top liner because a portion D of leakage E is absorbed by, or otherwise entrapped in, the top liner.

The following relationship exists between the four leakages discussed above:

$$J < H \ll E < C \qquad \text{(Equation 2.7-1)}$$

where < means smaller than and << much smaller than.

The amount of leakage, G, collected in the sump is given by the following equation:

$$G = E-H-F \qquad \text{(Equation 2.7-2)}$$

It is important that the leakage from the leak detection system, into the bottom liner (leakage H in Figure 2.7-1) be as small as possible for two reasons:

- to make sure that leakage into the ground (leakage J in Figure 2.7-1) is very small; and
- to ensure proper monitoring of the performance of the top liner by making sure that leakage rate measured at the sump (i.e., E-H-F) is as close as possible to leakage E through the top liner.

A bottom liner that leaks very much (i.e., large H) would give the illusion that the lining system functions well (i.e., small value of observed leakage E-H-F), while allowing significant ground pollution (i.e., large value of J).

Leakage from the leak detection system into the bottom liner (i.e., H in Figure 2.7-1) is small if the following conditions are met:

- the depth of liquid in the leak detection system is small (i.e., the hydraulic head acting on the bottom liner is small);

- the bottom liner is a FML-compacted soil composite liner, which absorbs much less liquid than a compacted soil alone (see the EPA Background Document on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments", USEPA, 1987); and

- extensive quality assurance is performed during the construction of the two components of the bottom composite liner, the soil and the FML, to ensure no major defects in either component.

The first of the three above conditions, a small liquid depth in the leak detection system, requires the following:

- leakage E through the top liner should be small (which is better achieved if the top liner is a composite liner than a FML alone);

- the leak detection system should be adequately designed with high hydraulic conductivity materials to ensure small liquid depth; and

- the leak detection system should be properly constructed, with clean draining materials to prevent clogging, and appropriate slopes to prevent ponding of the collected leakage in the leak detection system.

The last requirement implies that construction quality assurance of lining systems should not be restricted to liner materials, but should also be concerned with the construction of the drainage layers. The first requirement (small leakage E) requires that leakage C be small, which in turns requires that, in the case of landfill, the leachate depth in the leachate collection and removal system above the top liner be small. This further emphasizes the importance of proper design and quality assurance of drainage layers.

### 2.7.4.2.2 Evaluation of Leachate Collection Efficiency

#### - Expression of Leachate Collection Efficiency

Using the definition presented in Section 2.7.4.1.2 and the analysis presented in Section 2.7.4.2.1, the cumulative leachate collection efficiency (measured from the time of unit startup to any other point in time) can be expressed as follows:

$$CLCE = G/E \qquad \text{(Equation 2.7-3)}$$

Combining Equations 2.7-2 and 2.7-3 gives:

$$CLCE = 1 - (H + F)/E \qquad \text{(Equation 2.7-4)}$$

where: CLCE = cumulative leachate collection efficiency; G = cumulative amount of leakage collected at the leak detection system sump; E = cumulative leakage through the top liner (i.e., leakage which enters the leak detection system); H = cumulative leakage from the leak detection system into the bottom liner; and F = cumulative leakage that remains entrapped in the leak detection system. The cumulative leachate collection efficiency, CLCE, is dimensionless, while G, E, H and F should be expressed in liters/1000m$^2$ or gallons/acre.

Another way of defining collection efficiency is the steady-state leakage collection efficiency (SSLCE). The determination of the SSLCE assumes a constant rate of leakage from the top liner into the LDCRS and from the LDCRS into the bottom liner. Further, under steady-state conditions the quantity of liquid held in storage in the LDCRS is assumed to be constant. For this set of assumptions, the SSLCE is given by:

$$SSLCE = 1 - H/E \qquad \text{(Equation 2.7-5)}$$

where: H and E can be expressed in units of liters/1000m$^2$/day (Ltd) or gallons/acre/day (gpad), while SSLCE is dimensionless.

- Discussion

Table 2.7-3 provides SSLCE values for several different lining systems. From Table 2.7-3, it can be observed that the SSLCE of land disposal units with composite bottom liners are better than units with compacted soil bottom liners. For this reason, as well as for the better detection sensitivities obtained with composite bottom liners composites are strongly recommended for the bottom liners at land disposal units.

- Conclusion

Two leachate collection efficiencies have been identified: the steady-state value (SSLCE) and the cumulative value (CLCE). These two values will be similar if the leachate stored in the LDCRS is small (if F is small in Figure 2.7-1) and if the bottom liner minimizes leakage into it (if H is small in Figure 2.7-1). Both of these conditions are met by the leak detection system proposed in this background document. First, leachate stored in the LDCRS is minimized by using a drainage medium in the LDCRS with minimal capillarity. As will be seen in Section 2.8 of this report, by using a drainage medium with a minimum hydraulic conductivity of $10^{-2}$ m/s (1 cm/s), capillarity will be minimal. Second, leakage into the bottom liner is minimized through the use of a composite bottom liner. Since these two conditions will be met in practice, it is acceptable to consider the steady-state leachate collection efficiency (which is easy to calculate) rather than the cumulative leachate collection efficiency (which is more difficult to calculate).

The steady-state leachate collection efficiency (SSLCE) is very sensitive to the type of bottom liner in the land disposal unit. This fact can be seen in Table 2.7-3, where it is shown that a certain landfill with a compacted soil bottom liner has a SSLCE of 45%, while in contrast, the same landfill with a composite bottom liner has a SSLCE of 99.9%. Clearly, the SSLCE highlights differences in the performance of compacted soil and composite bottom liners. However, if only composite bottom liners are considered, which is the case for the leak detection systems proposed in this background document, the SSLCE will be very high for all cases except those where the rate of top liner leakage is insignificantly small. Since the SSLCE will be very high for all of the leak detection systems proposed in this document, there is no need to explicitly require SSLCE as a performance criterion.

Table 2.7-1. Leakage rates through FML and composite top liners. The small hole has a diameter of 2 mm (0.08 in.). The standard hole has a surface area of 1 cm² (0.16 in²). The frequency of holes is 1 per 4,000m² (1 per acre). The thickness of the compacted soil layer is 0.9 m (3 ft) and its hydraulic conductivity is $10^{-8}$ m/s ($10^{-6}$ cm/s). Note: Ltd = liter/1000m²/day; gpad = gallons/acre/day; 1 Ltd = 1.1 gpad. This table is a reproduction of Table 2.2-16 from Section 2.2.

| | | Hydraulic head, h | | |
|---|---|---|---|---|
| Type of Liner | Leakage mechanism | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| FML alone | Permeation<br>Small hole | 0.001<br>30 | 0.1<br>100 | 10<br>300 |
| | TOTAL | 30 | 100 | 300 |
| | Permeation<br>Standard hole | 0.001<br>300 | 0.1<br>1,000 | 10<br>3,000 |
| | TOTAL | 300 | 1,000 | 3,000 |
| Composite liner (good contact) | Permeation<br>Standard hole | 0.001<br>0.02 | 0.1<br>0.2 | 10<br>3 |
| | TOTAL | 0.02 | 0.3 | 13 |
| Composite liner (poor contact) | Permeation<br>Standard hole | 0.001<br>1 | 0.1<br>8 | 10<br>50 |
| | TOTAL | 1 | 8 | 60 |
| | | Values of leakage rate in Ltd or gpad | | |

Table 2.7-2. Leak detection time as a function of the distance, L, between the leak and the collector pipe, and the hydraulic conductivity of the leak detection system material. This table has been established assuming steady-state flow in the leak detection system. The slope of the leak detection system is 2%. This table is a reproduction of Table 2.6-1. [Result of the two-dimensional analytical study presented in Section 2.6.2.]

| L | Hydraulic conductivity of the leak detection system material, $k_d$ | | | |
|---|---|---|---|---|
| | $10^{-4}$ m/s ($10^{-2}$ cm/s | $10^{-3}$ m/s ($10^{-1}$ cm/s) | $10^{-2}$ m/s (1 cm/s) | $10^{-1}$ m/s (10 cm/s) |
| 15 m (50 ft) | 26 | 3 | 0.3 | 0.03 |
| 30 m (100 ft) | 52 | 5 | 0.5 | 0.05 |
| 50 m (165 ft) | 87 | 9 | 0.9 | 0.01 |
| 60 m (200 ft) | 104 | 11 | 1.1 | 0.1 |
| 100 m (330 ft) | 174 | 18 | 2 | 0.2 |

Leak detection times in days

Table 2.7-3. Examples of steady-state leachate collection efficiencies (SSLCE). Values for leakage through the top liner (E) and into the bottom liner (H) are obtained from Table 2.7-1. G and SSLCE were calculated using Equations 2.7-2 and 2.7-4. The various types of leakage, E, H, and G, are illustrated in Figure 2.7-1.

| | | Various leakages as defined in Figure 2.7-1 in Ltd (gpad) | | | Steady State Leachate Collection Efficiency |
|---|---|---|---|---|---|
| Case | Description | E | H | G | SSLCE |
| (1) | Landfill with FML top liner and compacted soil bottom liner. | 200 | 90 | 110 | 0.45 |
| (2) | Landfill with FML top liner and composite bottom liner. | 200 | 0.2 | 199.8 | 0.999 |
| (3) | Landfill with composite top liner and compacted soil bottom liner. | 1 | 1 | 0 | 0 |
| (4) | Landfill with composite top liner and bottom liner. | 1 | 0.2 | .8 | 0.80 |
| (5) | Surface impoundment with composite top liner and composite bottom liner. | 20 | 0.2 | 19.8 | 0.99 |

| | |
|---|---|
| Leachate Collection and Removal System (LCRS) | A = leachate collected in the LCRS<br>B = leachate stored* in LCRS<br>C = leachate from the LCRS into top liner |
| Top Liner | D = leachate stored* in top liner<br>E = leakage from the top liner into the LDCRS |
| Leak Detection Collection and Removal System (LDCRS) | G = leakage collected in the LDCRS sump<br>F = leakage stored* in LDCRS<br>H = leakage from the LDCRS into the bottom liner |
| Bottom Liner | I = leakage stored* in the bottom liner |
| Ground | J = leakage from the bottom liner into the ground |

* Stored liquids due to capillarity, absorption, etc.

Figure 2.7-1. Fate of liquids entering a double liner system at a landfill unit. Thie Figure is a reproduction of Figure 2.1-3.

# 2.8 DESIGN SPECIFICATIONS FOR LEAK DETECTION SYSTEMS

## 2.8.1 Introduction

### 2.8.1.1 Scope of the Section

#### 2.8.1.1.1 Purpose of the Section

The purpose of this section is to review the technical information pertinent to design specifications for leak detection systems. This information supports:

- the selection of the leak detection system design parameters that are most appropriate to establish design specifications; and

- the establishment of the specifications themselves, which are expressed as values (maximum or minimum, depending on the considered parameter) that must be met by the selected design parameters.

#### 2.8.1.1.2 Organization of the Section

This section is organized as follows:

- The remainder of this introduction discusses the concept of "design specifications".

- Section 2.8.2 reviews the leak detection system design parameters that can be considered to establish design specifications, and selects those that are deemed appropriate.

- Section 2.8.3 reviews the technical information relevant to the various design specifications and the rationale for the establishment of these specifications.

- Section 2.8.4 presents and discusses the design specifications.

### 2.8.1.2 The Concept of Design Specifications

#### 2.8.1.2.1 Definition

Design specifications are requirements that selected physical characteristics of the leak detection system should meet. Examples of such characteristics are the dimensions of the leak detection system and properties of the materials used to construct the system. These physical characteristics which are selected to establish design specifications are those which govern the design and hereafter they are called design parameters.

#### 2.8.1.2.2 Demonstration

Demonstration by the owner that a leak detection system meets design specifications is done in two stages:

- at the design stage, the owner should select, and write project specifications for, leak detection system geometry and materials that meet the design specifications; and
- at the construction stage, conformance testing and measurements should be made to verify that the materials used in the leak detection system and the geometry of the leak detection system meet the design specifications and agree with the design.

#### 2.8.1.2.3 Usefulness of Design Specifications

A comparison between performance criteria and design specifications is presented in Section 2.7.1.2.3. From this comparison, the following conclusions can be drawn regarding the usefulness of design specifications:

- Special expertise is required to design a leak detection system and to demonstrate that it will meet performance criteria and provide adequate leak detection capability.

- Special expertise is also required to evaluate such designs and demonstrations.

- Because of the increasing use of synthetic drainage layers and the possible growing use of other innovative technologies, it is expected that some designers, reviewers, and permit writers will, at least initially, lack expertise in these technologies.

- Guidelines on leak detection system design are therefore needed by designers, reviewers, and permit writers.

- The simplest guidelines are design specifications. They are simple to follow and their implementation is simple to verify.

It may be concluded that design specifications are necessary safeguards. Without them, mistakes could be made at the design stage, and many would remain undetected.

#### 2.8.1.2.4 Conservativeness of Design Specifications

When a design must meet performance criteria and satisfy design specifications, the design specifications do not need to be conservative because their role, in this case, is only to act as safeguards. For such cases, it may even be preferable to have design specifications that are not conservative in order to allow designers latitude for innovation, provided they can demonstrate those innovative designs meet the performance criteria.

Design specifications must be conservative in those simple applications where no real design is performed and where the only performance criterion is that "it should work".

## 2.8.2 Selection of Design Parameters

### 2.8.2.1 Introduction

#### 2.8.2.1.1 Purpose

To be adequate, design specifications must include the design parameters which have an important influence on the performance of the system. It is therefore essential that design parameters which will be used to establish design specifications be carefully selected.

#### 2.8.2.1.2 Method

Equations, charts, and numerical results pertinent to the performance of leak detection systems should be reviewed to determine which parameters are important. As discussed in Section 2.7, the essential performance characteristics are detection sensitivity and detection time. Leak detection systems are also used as leachate collection and removal systems between the liners. As such, their relevant performance characteristic is the liquid head on the bottom liner, which must be small. Consequently, equations, charts, and numerical results related to detection sensitivity, detection time, and liquid head should be reviewed.

### 2.8.2.2 Review of Design Parameters

#### 2.8.2.2.1 Review of Parameters Governing Detection Sensitivity

As discussed in Section 2.7.2, the required detection sensitivity can be achieved only if leakage into or through the bottom liner is very small. Such a requirement cannot be achieved by a compacted soil bottom liner as discussed in the EPA "Background Document on Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments" (USEPA, 1987). Therefore, a composite liner should be used, and the design specifications regarding composite bottom liners discussed in the above referenced document are useful to ensure proper functioning of the leak detection system. Bottom liner specifications will not be discussed hereafter. The design parameters discussed hereafter are specific to the leak detection system.

### 2.8.2.2.2 Review of Parameters Governing Detection Time

According to the two-dimensional analytical study presented in 2.6.2, detection time depends on the following parameters which govern steady-state flow: hydraulic conductivity of the leak detection system material, slope of the leak detection system, and distance between the leak and the collector pipe or sump. Short detection times result from high hydraulic conductivities, steep slopes, and short distances between leaks and collector pipes.

Leak detection system materials with a high hydraulic conductivity exhibit little capillary rise and, therefore, short initial detection time, which is an additional benefit.

### 2.8.2.2.3 Review of Parameters Governing Liquid Head

According to the study presented in Section 2.6.2.3.3, the liquid head on the bottom liner is governed by the following parameters: the hydraulic conductivity of the leak detection system material, the slope of the detection system, and the distance between the leak and the collector pipe. Small heads are obtained if:

- hydraulic conductivity is large;
- slope is steep; and
- distance between leak and collector pipes is small.

However small the liquid head is, it is never zero and the leak detection system thickness should be greater than the head.

### 2.8.2.3 Design Parameters to be Considered in Specifications

From the above discussion it appears that the relevant design parameters are:

- the hydraulic conductivity of the leak detection system material;

- the thickness and the slope of the leak detection system; and
- the distance between the leak and the collector pipe or sump.

From the above discussion it also appears that the leak detection system will perform better (i.e., the detection time will be shorter and the head smaller) if:

- the hydraulic conductivity is large;
- the slope is steep; and
- the distance between the leak and the collector pipe or sump is short.

In addition, the leak detection system thickness must be large enough to contain the head required by the flow.

The values that these parameters should have are discussed in the following sections.

Thickness, D, of the leak detection system, and hydraulic conductivity, $k_d$, of the leak detection system material can be combined to give the hydraulic transmissivity, $\theta_d$ (with $\theta_d = k_d D$), of the leak detection system. The hydraulic transmissivity can be used as a design parameter for granular as well as synthetic drainage layers. However, it is particularly convenient for synthetic drainage layers for which hydraulic transmissivity is directly obtained from tests.

### 2.8.3 Establishment of Design Specifications

#### 2.8.3.1 Summary of Relevant Data

##### 2.8.3.1.1 Performance Criteria to Meet

From Section 2.7, the performance criteria to meet are as follows:

- detection sensitivity: 1 liter/1000m²/day (1 gpad); and
- detection time: 1 day.

### 2.8.3.1.2 Relevant Technical Information for Design Specifications

- Available Technical Information

Available technical data which can be used to establish leak detection system design specifications can be summarized in four tables:

- Table 2.8-1 gives detection times as a function of the distance between the leak and the collector pipe or sump for various hydraulic conductivities of leak detection system material. This table (which is a reproduction of Table 2.6-1 established considering steady-state flow governed by Darcy's equation) illustrates the influence of leak detection system material hydraulic conductivity on detection time.

- Table 2.8-2 gives the values of capillary rise in the leak detection system as a function of the hydraulic conductivity of the drainage medium used to construct the leak detection system. This table is a reproduction of Table 2.6-4.

- Table 2.8-3 gives required hydraulic transmissivities as a function of the leakage rate and the distance between the leak and the collector pipes or sump. This table (which is a reproduction of Table 2.6-2) has been established considering a steady-state flow governed by Darcy's equation for several very large leakage rates. A leakage rate of 10 000 liters/1000m²/day (10,000 gallons/acre/day) represents a rapid and extremely large leakage (RLL) with a factor of safety.

- Table 2.8-4 gives the required thickness, D, for a leak detection system as a function of the hydraulic conductivity, $k_d$, of the leak detection system material, the slope of the leak detection system, and the distance, L, between the leak and the collector pipe. This table (which is a reproduction of Table 2.6-3) has been established considering steady-state flow governed by Darcy's equation.

- Additional Technical Information

As indicated in Section 2.8.2.1.2, the hydraulic head on the bottom liner should be as small as possible and, therefore, it is important that the characteristics of a leak detection system be such that the hydraulic head on the bottom liner be as small as possible.

The hydraulic head on the bottom liner can be approximated using Moore's equation [Moore, 1983] which assumes that the leak is uniformly distributed:

$$h = L \left[\sqrt{(q/k_d) + \tan^2 \beta} - \tan \beta\right] \qquad \text{(Equation 2.8-1)}$$

where: $h$ = hydraulic head on top of the bottom liner; $L$ = length of the considered area impinged by a uniform leak (see Figure 2.6-1); $q$ = uniform leakage rate; $k_d$ = leak detection system material hydraulic conductivity; and $\beta$ = slope of the leak detection system. Recommended SI units are: $h$ (m), $L$ (m), $q$ (m/s), and $k_d$ (m/s).

It should be pointed out that Moore's equation is related to steady-state flow, which is consistent with the other technical data related to leak detection system performance criteria.

Table 2.8-5 has been established using Moore's equation for a leakage rate of 10,000 liters/1000m²/day (10,000 gallons/acre/day), which represents a rapid and extremely large leak with a factor of safety. (A rapid and extremely large leakage may be on the order of 1,000-5,000 Ltd (gpad).)

### 2.8.3.2 Rationale for the Specifications

- Hydraulic Conductivity and Length of Leak Detection System

Table 2.8-1 shows that a detection time equal to or less than one day can be achieved only if the leak detection system hydraulic conductivity is equal to or greater than $10^{-2}$ m/s (1 cm/s).

In addition, Table 2.8-1 shows that the distance between the leak and the collector pipe or sump should be less than 50 m (165 ft) in order to meet the performance criterion of detection time less than one day. To ensure that the distance between the leak and the collector pipe or sump be less than 50 m (165 ft), the length of the leak detection system should be smaller than 50 m (165 ft).

Table 2.8-2 shows that a drainage medium with a hydraulic conductivity larger than $10^{-2}$ m/s (1 cm/s) exhibits a small capillary rise (i.e., 30 mm (1.2 in.)). Therefore, a leak detection system constructed with such a material will not retain excessively large amounts of leakage by capillarity. This confirms that $10^{-2}$ m/s (1 cm/s) should be considered as a minimum for the hydraulic conductivity of the leak detection system.

It is important to remember that Table 2.8-1 is for a bottom slope of 2%. For a detection time of one day, a longer drainage path is possible if a steeper bottom slope is incorporated into the design. Alternatively, for a given length of drainage path, the detection time can be decreased by steepening the bottom slope.

- Hydraulic Transmissivity

The required hydraulic transmissivity of a leak detection system depends on the considered leakage rate. As discussed before, a leakage rate on the order of 10,000 Ltd (gpad) could be considered in order to handle a rapid and extremely large leakage (RLL) with a factor of safety. Table 2.8-3 shows that for such a leakage rate a hydraulic transmissivity of $1 \times 10^{-4}$ m$^2$/s would be too small. A hydraulic transmissivity of $3 \times 10^{-4}$ m$^2$/s is required for a 50 m

(165 ft) distance between the leak and the collector pipe. For increased safety, a value of $5 \times 10^{-4}$ m²/s is recommended.

- Thickness

The hydraulic transmissivity, $\theta_d$, the thickness, D, and the hydraulic conductivity, $k_d$, of the leak detection system are related through the following equation:

$$\theta_d = D\, k_d \qquad \text{(Equation 2.8-2)}$$

Using Equation 2.8-2, it appears that the requirements indicated above for hydraulic conductivity (i.e., $k_d \geq 1 \times 10^{-2}$ m/s (1 cm/s)) and hydraulic transmissivity (i.e., $\theta_d \geq 5 \times 10^{-4}$ m²/s) can be met as follows by the leak detection system:

- either $k_d \geq 10^{-1}$ m/s (10 cm/s) and $D \geq 5$ mm (0.2 in.);
- or $k_d \geq 10^{-2}$ m/s (1 cm/s) and $D \geq 50$ mm (2 in.).

The first set of design specifications is appropriate for a synthetic drainage layer, while the second set of specifications is appropriate for a granular material.

In fact, the minimum thickness of 50 mm (2 in.) for a granular leak detection system should be replaced by a larger thickness such as 300 mm (12 in.) for the following reasons: (i) placement of a soil layer much less than 300 mm (1 ft) is not practical; and (ii) the construction equipment used to spread granular materials should not operate on top of the FML component of the bottom liner unless the FML is covered by a protective layer approximately 300 mm (1 ft) thick.

The minimum thickness values mentioned above are confirmed by Table 2.8-5 which gives the hydraulic head in the leak detection system for a rapid and extremely large leakage with a factor of safety. (Note: hydraulic heads indicated in Table 2.8-5 are smaller than the thicknesses given in Table 2.8-4 because Moore's equation considers a leakage uniformly distributed over the entire surface area of the leak detection system, which is not conservative, while calculations performed in Section 2.6-2 and leading to Table 2.8-4 consider leakage concentrated at the top of the leak detection system.)

- Slope

All design specifications discussed above were related to a leak detection system with a 2% slope. Tables 2.8-4 and 2.8-5 show that a 4% slope would clearly result in smaller hydraulic heads on the bottom liner. These tables also show that hydraulic heads obtained with a 2% slope are not excessive and, therefore, a 2% slope is theoretically appropriate.

However, field experience shows that bottom liners which are designed with a 2% slope are often constructed with a slope that is locally less than 2%. Also, settlement can locally reduce the slope after waste has been placed. In well designed hazardous waste management units, allowance is made for settlement and the 2% slope is the "after-settlement slope". However, since design specifications are mostly intended as guidance to prevent mistakes in cases where design is insufficient and/or construction quality is below standards, a 4% slope design specification could be considered.

## 2.8.4 Presentation and Discussion of the Design Specifications

### 2.8.4.1 Presentation of the Design Specifications

The design specifications can be summarized as follows:

- Hydraulic conductivity of leak detection system material: $k_d \geq 10^{-2}$ m/s (1 cm/s).

- Hydraulic transmissivity of leak detection system (regardless of type): $\theta_d \geq 5 \times 10^{-4}$ m²/s.

- Thickness of geosynthetic leak detection system: $D \geq 5$ mm (0.2 in.), which implies a minimum hydraulic conductivity of $10^{-1}$ m/s (10 cm/s) for the geosynthetic material in order to meet the above hydraulic transmissivity requirement.

- Thickness of granular leak detection system: $D$ [illegible] 0.3 m (1 ft).

- Slope of leak detection system (including pipes): $\tan \beta \geq 0.02$ (2%), with a recommendation that a 4% slope be considered.

- Length of leak detection system: $L \leq 50$ m (165 ft).

### 2.8.4.2 Comments

- Comments on Hydraulic Transmissivity

It is essential that the hydraulic transmissivity (and the two related properties, hydraulic conductivity and thickness) of the leak detection system be evaluated by tests conducted with boundary conditions (such as compressive stress and materials in contact) representative of conditions at the considered site. This recommendation is so important that it should not be considered as a simple comment but should be included in the specifications.

- Comments on Slope

As indicated in Section 2.8.4.1, a 2% slope is theoretically appropriate, while a 4% slope would be advisable in some hazardous waste management units to compensate for insufficient design regarding settlement and/or poor grade control during construction. To select between 2% and 4% the following strategy may be considered: the specified slope would be 4% unless: (i) the owner or operator can demonstrate that the design slope is such that after settlement the actual slope will be at least 2% everywhere; and (ii) the quality assurance plan includes a detailed survey to ensure that the bottom liner will be constructed with the design slope.

Table 2.8-1. Leak detection time as a function of the distance, L, between the leak and the collector pipe or sump, and the hydraulic conductivity of the leak detection system material. This table has been established assuming steady-state flow in the leak detection system. The slope of the leak detection system is 2%. This table is reproduced from Table 2.6-1 in Section 2.6.

| L | Hydraulic conductivity of the leak detection system material, $k_d$ | | | |
|---|---|---|---|---|
| | $10^{-4}$ m/s ($10^{-2}$ cm/s | $10^{-3}$ m/s ($10^{-1}$ cm/s) | $10^{-2}$ m/s (1 cm/s) | $10^{-1}$ m/s (10 cm/s) |
| 15 m (50 ft) | 26 | 3 | 0.3 | 0.03 |
| 30 m (100 ft) | 52 | 5 | 0.5 | 0.05 |
| 50 m (165 ft) | 87 | 9 | 0.9 | 0.01 |
| 60 m (200 ft) | 104 | 11 | 1.1 | 0.1 |
| 100 m (330 ft) | 174 | 18 | 2 | 0.2 |
| | Leak detection times in days | | | |

Table 2.8-2. Values of capillary rises as a function of the hydraulic conductivity of the drainage medium used in the leak detection system. This table is reproduced from Table 2.6-4 in Section 2.6.

| Hydraulic Conductivity $k_d$ |  | Capillary Rise $h_c$ |  |  |
|---|---|---|---|---|
| (m/s) | (cm/s) | (m) | (mm) | (in.) |
| $1 \times 10^{-6}$ | $1 \times 10^{-4}$ | 3.10 | 3100 | 122 |
| $5 \times 10^{-6}$ | $5 \times 10^{-4}$ | 1.39 | 1386 | 54.6 |
| $1 \times 10^{-5}$ | $1 \times 10^{-3}$ | 0.98 | 980.3 | 38.6 |
| $5 \times 10^{-5}$ | $5 \times 10^{-3}$ | 0.44 | 438.4 | 17.3 |
| $1 \times 10^{-4}$ | $1 \times 10^{-2}$ | 0.31 | 310.0 | 12.2 |
| $5 \times 10^{-4}$ | $5 \times 10^{-2}$ | 0.14 | 138.6 | 5.5 |
| $1 \times 10^{-3}$ | $1 \times 10^{-1}$ | 0.10 | 98.0 | 3.9 |
| $5 \times 10^{-3}$ | $5 \times 10^{-1}$ | 0.044 | 43.8 | 1.7 |
| $1 \times 10^{-2}$ | 1 | 0.031 | 31.0 | 1.2 |
| $5 \times 10^{-2}$ | 5 | 0.014 | 13.9 | 0.55 |
| $1 \times 10^{-1}$ | 10 | 0.010 | 9.8 | 0.39 |
| $5 \times 10^{-1}$ | 50 | 0.004 | 4.4 | 0.17 |
| 1 | 100 | 0.003 | 3.1 | 0.12 |
| 5 | 500 | 0.0014 | 1.4 | 0.055 |

Table 2.8-3. Required hydraulic transmissivity, $\theta_d$, for the leak detection system (in $m^2/s$) as a function of leakage rate, q, and distance, L, between leak and collector pipe for a leak detection system slope of 2%. This table is reproduced from Table 2.6-2 in Section 2.6.

| L | Leakage rate per unit area, q liters/1000m²/day (gallons/acre/day) | | |
|---|---|---|---|
| | 1000 | 10 000 | 100 000 |
| 15 m (50 ft) | $1.2 \times 10^{-5}$ | $1.2 \times 10^{-4}$ | $1.2 \times 10^{-3}$ |
| 30 m (100 ft) | $1.8 \times 10^{-5}$ | $1.8 \times 10^{-4}$ | $1.8 \times 10^{-3}$ |
| 50 m (165 ft) | $2.9 \times 10^{-5}$ | $2.9 \times 10^{-4}$ | $2.9 \times 10^{-3}$ |
| 60 m (200 ft) | $3.6 \times 10^{-5}$ | $3.6 \times 10^{-4}$ | $3.6 \times 10^{-3}$ |
| 100 m (330 ft) | $5.8 \times 10^{-5}$ | $5.8 \times 10^{-4}$ | $5.8 \times 10^{-3}$ |
| | Required hydraulic transmissivity for the leak detection system, $\theta_d$, in $m^2/s$ | | |

Table 2.8-4. Required thickness, D, for a leak detection system as a function of hydraulic conductivity, $k_d$, of the leak detection system material, the slope of the leak detection system, and the distance, L, between the leak and the collector pipe. These results are related to a leakage rate of 10,000 liters/1000m²/day (10,000 gallons/acre/day). (Note: this table is a reproduction of Table 2.6-3 from Section 2.6.)

| | | Hydraulic conductivity, $k_d$ | | |
|---|---|---|---|---|
| Slope | - | 0.01 m/s<br>(1 cm/s) | 0.1 m/s<br>(10 cm/s) | 1 m/s<br>(100 cm/s) |
| 2% | 15 m<br>(50 ft) | 9 mm<br>(0.3 in.) | 0.9 mm<br>(0.03 in.) | 0.1 mm<br>(0.003 in.) |
| | 60 m<br>(200 ft) | 35 mm<br>(1.3 in.) | 3.5 mm<br>(0.13 in.) | 0.3 mm<br>(0.013 in.) |
| | 100 m<br>(330 ft) | 58 mm<br>(2.5 in.) | 5.8 mm<br>(0.25 in.) | 0.6 mm<br>(0.025 in.) |
| 4% | 15 m<br>(50 ft) | 4 mm<br>(0.17 in.) | 0.4 mm<br>(0.017 in.) | 0.04 mm<br>(0.0017 in.) |
| | 60 m<br>(200 ft) | 17 mm<br>(0.64 in.) | 1.7 mm<br>(0.064 in.) | 0.17 mm<br>(0.006 in.) |
| | 100 m<br>(330 ft) | 29 mm<br>(1.3 in.) | 2.9 mm<br>(0.13 in.) | 0.3 mm<br>(0.013 in.) |

Table 2.8-5 Liquid head on the bottom liner as a function of the leak detection system hydraulic conductivity, $k_d$, the slope, and the distance, L, between the leak and the collector pipe or sump. This table was established using Moore's equation [Moore, 1983] for a leakage rate of 10,000 Ltd (gpad).

| | | Hydraulic conductivity, $k_d$ | | |
|---|---|---|---|---|
| Slope | L | 0.01 m/s<br>(1 cm/s) | 0.1 m/s<br>(10 cm/s) | 1 m/s<br>(100 cm/s) |
| 2% | 15 m<br>(50 ft) | 4.3 mm<br>(0.17 in.) | 0.43 mm<br>(0.017 in.) | .043 mm<br>(0.0017 in.) |
| | 30 m<br>(100 ft) | 8.6 mm<br>(0.3 in.) | 0.86 mm<br>(0.03 in.) | .086 mm<br>(0.003 in.) |
| | 60 m<br>(200 ft) | 17 mm<br>(0.7 in.) | 1.7 mm<br>(0.07 in.) | .017 mm<br>(0.007 in.) |
| | 100 m<br>(330 ft) | 29 mm<br>(1 in.) | 2.9 mm<br>(0.1 in.) | 0.29 mm<br>(0.01 in.) |
| 4% | 15 m<br>(33 ft) | 2.1 mm<br>(0.08 in.) | 0.21 mm<br>(0.008 in.) | 0.021 mm<br>(0.0008 in.) |
| | 30 m<br>(100 ft) | 4.3 mm<br>(0.2 in.) | 0.43 mm<br>(0.02 in.) | 0.043 mm<br>(0.002 in.) |
| | 60 m<br>(200 ft) | 8.7 mm<br>(0.3 in.) | 0.87 mm<br>(0.03 in.) | 0.087 mm<br>(0.003 in.) |
| | 100 m<br>(400 ft) | 14 mm<br>(0.6 in.) | 1.4 mm<br>(0.06 in.) | 0.14 mm<br>(0.006 in.) |
| | | Liquid head on the bottom liner | | |

# 2.9 ACTION LEAKAGE RATE (ALR)

## 2.9.1 Introduction

### 2.9.1.1 Purpose of the Section

The Action Leakage Rate (ALR) constitutes a trigger for initiating interactions between the owner or operator of a land disposal unit and EPA. Unlike the detection capability criteria described in Sections 2.7 and 2.8 of this document (which require the owner or operator to carry out a demonstration), the ALR is a standard that is compared to the leakage rates that the owner or operator measures (as part of a leak detection monitoring program) at the leak detection system sump. If the measured leakage rate exceeds the ALR (as described subsequently under monitoring requirements) the owner or operator will enter a response action mode and will be required to initiate interactions with the EPA Regional Administrator. In the proposed Liner/Leak Detection Rule owners and operators may elect to use a standard value of ALR defined by EPA or they may submit to the EPA for approval a site-specific ALR.

The purpose of this section is to review the technical information pertinent to the Action Leakage Rate. This information is used as a basis to support the rationale given in the proposed Liner/Leak Detection Rule for the establishment of the standard Action Leakage Rate.

### 2.9.1.2 Organization of the Section

This section includes three parts:

- the first part presents an overview of the concepts of the proposed rule regarding the Action Leakage Rate (ALR);
- the second part presents the available technical data that support the rationale for the establishment of the standard Action Leakage Rate; and

- the third part presents suggested leachate monitoring requirements.

## 2.9.2 Overview of the Concepts of the Proposed Rule

### 2.9.2.1 The Concept of a Trigger

The Action Leakage Rate triggers interactions between the owner or operator and the Regional Administrator (RA) and requires the owner or operator to submit a Response Action Plan (RAP) for leakage rates less than rapid and extremely large (RLL) or to immediately implement the RAP for leakage rates exceeding the RLL. This approach takes into account the characteristics of the double-liner system, is consistent with the capabilities of current technology, allows the owner or operator flexibility, and uses leachate volume rather than hazardous constituent concentrations as a trigger (except for land treatment units).

In developing the concept of a trigger for interactions between the owner or operator and the EPA, two different types of triggers could be considered: a hydraulic trigger based on the rate of leakage through the top liner, or a leakage quality trigger based on the concentrations of hazardous constituents in the leakage. It is believed that for a trigger mechanism, a hydraulic criterion has several distinct advantages over a leakage quality criterion. However, once the ALR is exceeded, hazardous constituent monitoring should be required as part of the response action activities. The advantages of a hydraulic trigger over a leakage quality trigger include:

- a measurement of leakage rate is more indicative of the magnitude and severity of a top liner breach than is a measurement of leachate quality;

- changes in leakage rate over time are indicative of progressive changes in the condition of the top liner; a knowledge that progressive changes are taking place is critical to selection of the appropriate response actions;

- the day-to-day monitoring program for a leakage rate trigger is fast, relatively inexpensive and can be conducted by maintenance personnel using relatively unsophisticated equipment; the day-to-day monitoring program for leakage quality would require complex, expensive chemical analyses, is more time consuming, and requires more highly trained personnel.

### 2.9.2.2 The Concept of Action Leakage Rate

The action leakage rate (ALR) logically extends EPA's systems approach to its liquids management strategy. An ALR not only initiates an interaction between an owner or operator, but also is a mechanism for the EPA to evaluate the leak detection program on a site-specific basis. The ALR and the Response Action Plan (discussed in Section 2.10) are key elements of an overall containment system. They function in an integrated interdependent manner with: (1) the top and bottom liners; (2) the leachate collection and removal system above the top liner (for landfills); (3) the leak detection, collection and removal system; and, where applicable, (4) the final unit cover.

The total system achieves the objective of preventing hazardous constituent migration from the unit by maximizing leachate collection and removal. The total system fails only if there is a fatal combination of failures among system components and if response actions triggered by the ALR are inappropriate or inadequate. The probability of a fatal combination of component failures is believed to be extremely low.

### 2.9.3 Technical Support for the ALR

#### 2.9.3.1 Introduction

- Purpose of the Section

The purpose of this section is to provide technical support for the issues of the proposed Liner/Leak Detection Rule related to the Action Leakage Rate (ALR).

- Organization of the Section

Section 2.9.3.2 reviews the available information. Then, the rationale for the Action Leakage Rate is provided in Section 2.9.3.3.

#### 2.9.3.2 Relevant Technical Information

The ALR is based on the technical capability of the top liner to prevent leakage through the top liners at properly designed and very well constructed units. The ALR therefore represents the best demonstrated available technology (BDAT) for top liners. (Note that the ALR is established independent of other possible sources of liquid in the leak detection system such as rainwater entrapped during construction or water expelled by consolidation from the soil component of the top composite liner. The way to account for these other sources of liquid is through an owner/operator demonstration as part of the response action plan.) The relevant technical information to use in establishing the ALR is the analyses presented in Section 2.2 on leakage through top liners and the information in Section 2.5 on existing land disposal units.

The technical information from Section 2.2 on leakage rates through top liners is summarized in Table 2.9-1. For the case of composite liners only the condition for good FML-compacted soil contact is shown since this represents the best demonstrated available technology (BDAT) for composite top liners. Table 2.9-1 gives leakage rates through top liners as a function of the hydraulic head on the liner. The three heads used in Table 2.9-1 represent the following conditions:

- 0.03 m (0.1 ft) is assumed to be the average head acting on the top liner of a landfill or waste pile with a well designed and constructed leachate collection and removal system (LCRS).
- 0.3 m (1 ft) is assumed to be maximum head acting on the top liner of a landfill or waste pile with a well designed and constructed leachate collection and removal system (LCRS).
- 3 m (10 ft) is assumed to be the maximum head acting on the top liner of a surface impoundment.

### 2.9.3.3 Rationale for the Action Leakage Rate Value

The owner or operator has the option in the proposed Liner/Leak Detection Rule of using an EPA-specified ALR which is generic and applicable to all land disposal units, or alternatively to use a site-specific ALR obtained after a site-specific demonstration. The site-specific ALR is intended to provide the owner or operator with a mechanism to account for conditions that reduce the potential for migration of hazardous constituents through the top liner such as the thickness of the top liner and its capacity to attenuate the specific hazardous constituents in the leachate at that site. The generic ALR, on the other hand, is based strictly on the best demonstrated available technology (BDAT) for hydraulic containment by the top liner. This section of the report presents the rationale for EPA's selection of a generic ALR in the range of 5-20 Ltd (gpad).

### - Leakage Due to Permeation Versus Holes

The ALR is based on best demonstrated available technology (BDAT) for leakage through top liners. This leakage can be due to two sources: permeation through the FML and leakage through defects (holes) in the FML. Since permeation will occur through all FMLs and is not indicative of a breach or defect in the top liner, it is not necessary to account for this source of liquid in the ALR determination. The permeation values presented in this report must be considered preliminary due to the limited data upon which they are based. Continuing research is providing a better understanding of permeation of liquids through FMLs. Once additional definitive permeation data is available, EPA may wish to consider allowing the owner or operator to subtract liquid associated with permeation through the FML when determining whether the liquid collected in the unit sump has exceeded the ALR. In conclusion, the ALR should be based solely on leakage through defects (holes) in the top liner.

### - Selection of a Value for ALR

Inspection of Table 2.9-1 shows that the leakage rate through a FML top liner will be dependent on the presence of FML defects and on the hydraulic head acting on the top liner. Best demonstrated available technology (BDAT) for FML top liners implies very high quality construction and intensive quality assurance. Therefore, BDAT should be based on an FML with, at most, only a small defect (rather than a standard defect which might be considered for design calculations). In fact, based on the very best installation techniques and very intensive construction quality assurance, holes smaller than 2 mm (0.08 in.) diameter occurring at frequencies less than 1 hole per 4,000 $m^2$ (acre) may be considered. If a range of one-1 mm (0.04 in.) diameter hole per 8,000 $m^2$ (2 acres) to one-2 mm (0.8 in.) diameter hole per 4,000 $m^2$ (1 acre) is considered to represent the range of BDAT for FML top liners, then the range for the ALR (assuming a 0.03 m (0.1 ft) hydraulic head on the top liner) would be from about 5 Ltd (gpad) to 30 Ltd (gpad). Choosing a value from the lower portion of this range, an ALR of 5 to 20 Ltd (gpad) may be reasonable.

The range of ALR calculated above is based on a hydraulic head of 0.03 m (0.1 ft). The reasonableness of this value for hydraulic head must be considered. In landfills and waste piles, the average hydraulic head on the top liner is expected to be about 0.03 m (0.1 ft) or less. The depth of liquid in the LCRS above the top liner will periodically be higher, due primarily to precipitation, and this will increase top liner leakage rates temporarily. However, the best way to account for these temporary increases is not to increase the ALR. The best way to account for these temporary increases is by using a time-weighted averaging procedure to process results from the monitoring program. The time-averaging process could be used to smooth out temporary increases in leakage rate due to precipitation so that a unit would not be found to be "leaking" at a rate exceeding the ALR each time a major storm occurred.

As can be deduced from Table 2.9-1, the range of ALR mentioned above is not appropriate for an FML top liner with holes in a surface impoundment with a 3 m (10 ft) depth. If the ALR were to be set at a level corresponding to an FML with holes, an ALR value in the range of 100 to 500 Ltd (gpad) would need to be considered in a surface impoundment. Clearly, allowing this much liquid to enter the leak detection system before initiating interactions between the owner/operator and EPA (to at least assess the consequences of this level of leakage) is undesirable because it would permit the buildup of high heads on the bottom liner.

However, the frequency of holes in operating surface impoundments should be smaller than the frequency of holes in landfills and waste piles. The reasons for this include: (i) hazardous waste surface impoundment units tend to be smaller than landfills and waste piles; (ii) there is less overburden pressure or construction equipment operating on top of liners in surface impoundments than in landfills and waste piles; (iii) because they are small and the consequences of holes are large, the CQA program for surface impoundments will often involve steps to identify and repair any holes (such as ponding tests and electrical resistivity surveys); and (iv) if a hole should occur during operation, repair or retrofitting of a surface impoundment top liner is often feasible. For the above reasons, it can be tentatively concluded that the frequency of holes in top liners of surface impoundments designed and constructed to BDAT standards can be less than the frequency at landfills and waste piles and may even approach zero defects (the "no holes" case in Table 2.9-1). This observation is consistent with the data on surface impoundments presented in Section 2.5. Based on the preliminary information presented in Section 2.5 for the ICWM surface impoundment (Section 2.5.2), the surface impoundment in East Central U.S. (Section 2.5.5.1) and the surface impoundments in South West U.S. (Section 2.5.5.2) it appears that top liner leakage rates of 5 to 20 Ltd (gpad) or less can be achieved and that the suggested ALR range represents BDAT for top liners at surface impoundments.

### 2.9.3.4 Monitoring Requirements

- Recommended Monitoring Interval

In order to determine whether the ALR has been exceeded (as well as to maintain a minimum hydraulic head in the leak detection system sump), the owner or operator should be required to institute a regular monitoring program at the leak detection system sump. The monitoring program will simply consist of taking regular measurements and keeping detailed records of the amount of liquid removed from the leak detection system sump.

Owners and operators should be encouraged to monitor the leak detection system as frequently as possible and as regularly as possible. Based on the leak detection system capabilities presented in Sections 2.7 and 2.8 a one-day monitoring interval is suggested during the active life and closure period of a land disposal unit. A longer monitoring interval during the post-closure care period is clearly acceptable. The recommendation for a one-day monitoring interval during the active life and closure period is based on three factors:

- The recommended maximum leak detection time for land disposal units is one day; the monitoring interval should be similar to the detection time in order to derive the full benefit (i.e., rapid leak detection) of this detection capability.

- Frequent monitoring allows the liquid level in the LDCRS sump to be kept to a minimum, which is essential to minimize leakage through that portion of the bottom liner underlying the sump.

- In the event of a top liner leakage rate in excess of the rapid and extremely large leakage rate (defined in Section 2.10), a short monitoring interval will minimize the length of time that the bottom liner of the land disposal unit could be subjected to significant hydraulic heads. In a one-day interval the leakage through a composite bottom liner subjected to 3.0 m (10 ft) of head (representing the worst case of a catastrophic failure of the top liner in a surface impoundment) would be small and within environmentally acceptable limits.

- ALR Determination

Having obtained daily leakage rate data from the monitoring of the LDCRS sump, the owner or operator must determine if the ALR has been exceeded. As noted previously, the daily leakage into the LDCRS will vary somewhat from day to day. Recent observations from active units indicate that even in the absence of precipitation leakage rates can easily vary by 10 to 20% or more. Much larger variations can be associated with major precipitation events. Due to these variations, it is appropriate to use time-weighted averages to determine if the ALR has been exceeded. Further, the period for conducting the time-weighted average should consider the maximum leakage rate measured. If a very high leakage rate is measured, the duration for the time-weighted average should be short so that the owner or operator moves quickly to begin interactions with EPA. EPA is recommending the following procedures to determine if the ALR has been exceeded (based on a ALR of 5 to 20 Ltd (gpad)):

- if no leakage rate measurement exceeds 50 Ltd (gpad) during the active life, average the measurements over 30 days to determine if the ALR has been exceeded;

- if no leakage rate measurement exceeds 350 liters/1000m²/week (gallons/acre/week) during the post-closure care period, average the measurements over 7 days to determine if the ALR has been exceeded; and

- if any leakage rate measurement exceeds 50 Ltd (gpad) during the active life, consider that the ALR has been exceeded.

These time-averaging procedures should be conducted on a daily, forward-rolling basis.

Table 2.9-1. Leakage rates through top liners. The small hole has a diameter of 2 mm (0.08 in.). The standard hole has a surface area of 1 $cm^2$ (0.16 $in.^2$). The frequency of holes is 1 per $4000m^2$ (1 per acre). The thickness of the compacted soil layer is 0.9 m (3 ft) and its hydraulic conductivity is $10^{-8}$ m/s ($10^{-6}$ cm/s). Good contact refers to the quality of the contact between the FML and the underlying low-permeability soil, which governs the flow between the FML and the low-permeability soil and, therefore, has a large influence on the leakage rate. Note: Ltd = liter/$1000m^2$/day; gpad = gallons/acre/day; 1 Ltd = 1.1 gpad. This table is adapted from results presented in Section 2.2.

| | | Hydraulic head, h | | |
|---|---|---|---|---|
| Type of Liner | Leakage mechanism | 0.03 m (0.1 ft) | 0.3 m (1 ft) | 3 m (10 ft) |
| FML alone | Permeation | 0.001 | 0.1 | 10 |
| | No hole | 0 | 0 | 0 |
| | TOTAL | 0.001 | 0.1 | 10 |
| | Permeation | 0.001 | 0.1 | 10 |
| | Small hole | 30 | 100 | 300 |
| | TOTAL | 30 | 100 | 300 |
| | Permeation | 0.001 | 0.1 | 10 |
| | Standard hole | 300 | 1,000 | 3,000 |
| | TOTAL | 300 | 1,000 | 3,000 |
| Composite liner (good) contact) | Permeation | 0.001 | 0.1 | 10 |
| | Small hole | 0.01 | 0.1 | 2 |
| | TOTAL | 0.02 | 0.2 | 12 |
| Composite liner (good) contact) | Permeation | 0.001 | 0.1 | 10 |
| | Standard hole | 0.02 | 0.2 | 3 |
| | TOTAL | 0.02 | 0.3 | 13 |
| | | Values of leakage rate in Ltd or gpad | | |

## 2.10 RESPONSE ACTION PLAN (RAP)

### 2.10.1 Introduction

#### 2.10.1.1 Scope

##### 2.10.1.1.1 The Response Action Plan (RAP)

As part of the implementation of EPA's leak detection standards, a written Response Action Plan (RAP) is proposed to be required of owners or operators of land disposal units. The RAP is a vehicle for site-specific response actions to be taken when leakage above the Action Leakage Rate (ALR) is detected within the leak detection system. The goal of the RAP is to prevent the migration of hazardous constituents out of the unit by providing a mechanism to initiate appropriate actions to mitigate the potential for such migration should the LDCRS reveal the presence of leakage above the ALR.

EPA is proposing RAPs for all newly constructed landfills, surface impoundments, and waste piles; for replacement landfill, surface impoundment and waste pile units; and for landfill and surface impoundment units required to have double liners after November 8, 1984, at both permitted and interim status facilities. RAPs are proposed for two leakage rates: (1) Rapid and extremely Large Leakage (RLL); and, (2) leakage rates less than the RLL but larger than the ALR. The Action Leakage Rate (ALR) was defined in Section 2.9. The RLL will be defined subsequently.

##### 2.10.1.1.2 Technical Support

The purpose of this section is to present technical information supporting the concept of Response Action Plan. Three areas have been identified where technical support is required: elements of the RAP, leakage bands, and sources of liquids other than leakage. These three areas are defined below.

- Technical Elements of the Response Action Plan

The first technical issue related to the RAP concerns the question of what information should be included in the RAP. The general answer to this question is that the RAP should include all information which would help the EPA, as well as the owner/operator, understand the design, construction, and operation of the land disposal unit, as well as the current performance of the unit and the anticipated future performance. The quantities and quality of leachate collected in the LDCRS should be documented. [Note: the quality of the leachate is not required in making an ALR determination; however, once the ALR is exceeded leachate quality must be evaluated.] The owner/operator should use this information in the RAP to develop a full range of response activities to actual or anticipated leakage events. The objective of any response activity is to ensure, to the extent feasible with current technology, that hazardous constituent migration out of the unit is prevented.

A more thorough discussion of the technical elements of the Response Action Plan are presented in Section 2.10.2.

- Leakage Bands

Since leakage through the top liner will fluctuate during a unit's active life and post-closure care period, it is logical to develop a RAP that has a range of leakage bands and a response or set of responses for each band. With a specific response tied to a leakage band instead of an individual leakage rate, the leakage rate can fluctuate over time without the need to implement a different resopnse for each small fluctuation. A leakage band refers specifically to a range of top liner leakage rates. For example, the owner or operator may recommend in a RAP the following leakage bands and response actions for a unit with an ALR of 20 Ltd (gpad) and a RLL of 2500 Ltd (gpad):

| Example of Leakage Bands | Typical Scope Responses |
|---|---|
| < 20 Ltd (gpad) | pump collected leachate |
| 20 Ltd (gpad) | notify Regional Administrator (ALR) |
| 20 - 250 Ltd (gpad) | increased pumping and monitoring of the leak detection system |
| 250 - 2,500 Ltd (gpad) | several changes in operating practices to reduce leakage to lower leakage band |
| > 2,500 Ltd (gpad) | repair leak or close unit (or part of unit) |

Leakage bands are site-specific and the values given above should only be considered simple examples. In this example, 20 Ltd (gpad) is the Action Leakage Rate (ALR) defined in Section 2.9, and 2,500 Ltd (gpad) is the Rapid and extremely Large Leak (RLL) defined in Section 2.10.3.

Technical support for the Action Leakage Rate was provided in Section 2.9. Section 2.10 provides technical support for the other elements of the leakage bands.

- Sources of Liquids Other than Leakage

Liquids collected in the leak detection system sump may come from sources other than leakage through the top liner, such as:

- rainwater, entrapped in the leak detection system during construction, which drains as soon as pumping of the LDCRS starts;

- water contained in the low-permeability soil component, if any, of the top liner, expelled as a result of compression of this layer under the pressure of the waste (a phenomenon known as consolidation); and

- ground water intruding into the leak detection system through the bottom liner.

If one or more of the above sources of water are present at the unit, the owner or operator has the opportunity to demonstrate that the leakage rate measurements have been disturbed by sources of liquids other than leakage through the top liner. If the Regional Administrator approves the demonstration, a variance to the Response Action Plan may be considered. Also, as noted in Section 2.9, the owner or operator may demonstrate that some small component of the top liner leakage is due to permeation through the FML. Usually, however, leakage associated with permeation is insignificant compared to other sources of liquid.

Technical support is presented in Section 2.10 regarding sources of liquids other than leakage through the top liner.

#### 2.10.1.2 Organization of the Section

Two issues requiring technical support have been identified above: leakage bands and sources of liquids other than leakage. Accordingly:

- Section 2.10.2 is devoted to a discussion of the necessary technical elements of the RAP;

- Section 2.10.3 is devoted to a technical discussion pertinent to leakage bands; and

- Section 2.10.4 is devoted to an evaluation of sources of liquids other than leakage.

### 2.10.2 Technical Elements of the Response Action Plan

As previously noted, the RAP should include all information that would help EPA and owners/operators understand the design construction and operation of the land disposal unit, as well as the current performance of the unit and its anticipated future performance. The quantities and quality of leachate collected in the LDCRS should be documented. The RAP should then go on to use this information to develop a full range of response activities to actual or anticipated leakage events. To achieve this end, it would appear that the RAP would need to contain, at a minimum:

(1) a general description of the unit;

(2) a description of the hazardous constituents contained in the unit;

(3) a description of the range of events that may potentially cause leakage (and the anticipated leakage rate associated with each event);

(4) a discussion of the factors that can affect the amount of liquid entering the LDCRS;

(5) a description of the design or operational mechanisms that can be used to prevent the migration of hazardous constituents out of the unit; and

(6) an assessment of the effectiveness of a range of possible response actions.

Each of these six technical elements of the RAP are briefly discussed below.

#### 2.10.2.1 General Description of Unit

The response action plan should include a general description of the unit including whether at closure the wastes will be decontaminated in place, removed from the unit, or left in place. The site-specific information should include, as a minimum, the type, size, and location of the unit; the design of the unit including details of the lining system; the geographic and climatic setting; and the operating history and practices at the unit including the age of the unit, planned unit active life, ongoing activities at the unit, volume of wastes being stored or disposed, methods of waste placement, equipment used, intermediate cover practices, and the closure plan.

#### 2.10.2.2 Hazardous Constituent Assessment

The response action plan should include a general discussion of the hazardous constituents contained in the unit. This discussion should include, at a minimum, a summary of the results of analyses carried out as part of the site-specific waste analysis plan (Sections 264.13(b) and 265.13(b)) as well as description of the physical characteristics of the waste. Of particular importance is the chemical quality of the leachate collected in the LDCRS and the compatibility of the waste in the unit and the leachate in the LDCRS with the lining system components.

#### 2.10.2.3 Description of Events Causing Leakage

The response action plan should include a discussion of all events that may potentially cause leakage exceeding both the ALR (if appropriate) and the RLL. These potential causes will be site-, design-, and operation-specific. In general, they may include operational accidents, design deficiencies identified subsequent to the start of unit operation (such as inadequate connections between liners and liner penetrations such as pipes and manholes), unforeseen incompatible wastes, equipment damage or unforeseen site subgrade settlements.

#### 2.10.2.4 Factors Influencing Liquid Quantities in the LDCRS

The response action plan should include a discussion of the important factors that can affect the amount of liquid entering the leachate collection and removal system between the liners. These factors should include, but not be limited to, the size and type of top liner breach, the potential for additional breaches in the future, the amount of liquid head in the leachate collection and removal system above the top liner, the potential for leachate generation in the unit due to the moisture content of the waste, the anticipated amount and frequency of precipitation, and the potential for surface-water run-on. The potential for sources of liquid other than top liner leakage should also be considered, including liquids from construction water, consolidation of any compacted soil component of the top liner, or water due to ground-water infiltration.

### 2.10.2.5 Mechanisms Preventing Migration Out of the Unit

The response action plan should include a description of the design and operational mechanisms that will prevent migration of hazardous constituents out of the unit. These mechanisms should consider the capabilities of the entire land disposal unit as well as the capability of each individual unit component. Particular attention should be given to: the condition of the composite bottom liner; the condition and operational capability of the leak detection system between the top and bottom liners; the condition and operational capabilities of the top liner and the leachate collection and removal system above the top liner; the potential to repair or retrofit the top liner if the RLL is exceeded; and the potential for the use of intermediate covers and run-on controls to limit leachate production potential in the unit.

### 2.10.2.6 Assessment of Response Actions

Last, the response action plan should include a detailed assessment describing the feasibility of each of a range of responses for preventing hazardous constituent migration out of the unit. For top liner leakage rates exceeding the RLL, the top liner leakage rate must be dramatically reduced in order to ensure, to the extent technically feasible, that hazardous constituent migration out of the unit will be prevented. It therefore appears that only a limited number of response action options are available. These options are:

(i) the owner or operator terminates receipt of waste and closes the unit (or part of the unit);

(ii) the owner or operator provides expeditious repair of the leak(s) (or retrofitting of the top liner); or

(iii) the owner or operator institutes operational changes at the unit that will reduce leakage into the space between the liners so that leakage will be less than rapid and extremely large.

In the case of leakage between the liners in excess of the ALR, but less than the RLL, the owner or operator has additional response action options. Therefore, the assessment in the RAP should include the three options listed above, plus:

(iv) the owner or operator continues to remove and treat the leakage with increased ground-water monitoring activities;

(v) the owner or operator maintains current operating procedures.

### 2.10.2.7 Sources of Information for the RAP

In developing the site-specific information for the response action plan, the owner or operator should evaluate the condition of the liners by reviewing activities that have occurred at the unit from the time of construction to the present. An analysis of the results of a rigorous construction quality assurance (CQA) plan should provide a good data base to assess the condition of the liners after construction of the unit. Results of CQA testing will be particularly valuable if key areas of the liner were tested hydraulically for leaks.

Other information that the owner or operator may use in development of a RAP includes: (1) the Part B permit application for the unit (for permitted units), (2) a review of operational records practices during the active life, (3) leachate analysis to indicate whether unanticipated waste constituents are present, (4) coupon testing in the sump above the top liner of a landfill or waste pile or in the waste at a surface impoundment to determine any chemical compatibility problems, and (5) an assessment of operating activities that may have damaged the liner. A review of the double liner system design can also reveal whether the design concept had any weaknesses that could increase the probability of a liner breach. The evaluation of the design will also indicate areas that include redundancy or design concepts that will minimize leakage if a breach occurs. This type of review of site-specific information may help isolate the location and extent of damage to a liner and can provide information showing that the breach is the result of a design, construction, or operational activity.

### 2.10.3 Leakage Bands

#### 2.10.3.1 Introduction

An example of leakage bands was given in Section 2.10.1.1.2. The two main elements of a set of leakage bands are:

- the Action Leakage Rate (ALR), which is the lower boundary of the set of leakage bands; and
- the Rapid and extremely Large Leakage (RLL), which is the upper boundary.

Technical support for the determination of the Action Leakage Rate has been provided in Section 2.9. Therefore, Section 2.10.2 will be entirely devoted to providing technical support for the Rapid and extremely Large Leakage (RLL).

#### 2.10.3.2 Rapid and Extremely Large Leakage

##### 2.10.3.2.1 Discussion

As intended for use in the proposed Liner/Leak Detection Rule, Rapid and extremely Large Leakage (RLL) is leakage that is equal to or larger than the maximum design leakage rate that the LDCRS can remove under gravity flow conditions (i.e., so that the fluid head on the bottom liner does not exceed the thickness of the LDCRS drainage medium, which is about 0.3 m (1 ft) for granular drainage materials and 5 mm (0.2 in.) for synthetic drainage materials). Leakage rates in excess of the RLL significantly increase the potential for migration of hazardous constituents into the bottom liner and out of the unit and therefore should be avoided. Remediation for RLL may include repair of the leak or closure of the unit.

### 2.10.3.2.2 Technical Support

- Scope

As a result of the above discussion, calculations should be carried out to determine typical leakage rates which can generate a 0.3 m (1 ft) hydraulic head on the bottom liner.

- Evaluation of Hydraulic Head

According to Giroud and Bonaparte [1984], the hydraulic head in a drainage layer, such as a leak detection system, with uniform flow is given by:

$$h = (Q_d/B)/(k_d \tan \beta) \qquad \text{(Equation 2.10-1)}$$

where: $h$ = hydraulic head; $Q_d$ = flow rate in the considered drainage layer; $B$ = width of the drainage layer perpendicular to the flow; $Q_d/B$ = flow rate per unit width; $k_d$ = hydraulic conductivity of the drainage medium; and $\beta$ = slope of the drainage layer. Recommended SI units are: $Q_d$ ($m^3/s$), $Q_d/B$ ($m^2/s$), $B$ (m), and $k_d$ (m/s).

In the case of a large leakage rate such as a Rapid and extremely Large Leakage (RLL), the width B is not the entire width of the leak detection system, but only the width, b, of the flow (Figure 2.10-1).

If N is the frequency of leaks:

$$q = N\,Q \qquad \text{(Equation 2.10-2)}$$

where: $q$ = leakage rate per unit area; $N$ = leak frequency; and $Q$ = leakage rate for one leak. Recommended SI units are: $q$ (m/s), $N$

($m^{-2}$), and $Q = m^3/s$. With the units used in this document: q is in Ltd or gpad; N is in number of holes per acre; and Q is in liters per day or gallons per day.

Combining Equation 2.10-1 and Equation 2.10-2 gives:

$$h = [q/(N\ b)]/(k_d \tan \beta)$$
$$= q/(N\ b\ k_d \tan \beta) \qquad \text{(Equation 2.10-3)}$$

where: b = width of the wetted area (b is used instead of B to prevent confusion with the width of the leak detection system).

If we consider the typical design case of one hole per 4,000 m² (1 hole per acre), $N = 1/4000\ m^2$ and Equation 2.10-3 becomes:

$$h = 4000\ q/(b\ k_d \tan \beta) \qquad \text{(Equation 2.10-4)}$$

with: h (m/s), q (m/s), b (m), and $k_d$ (m/s), or:

$$h = 4.6 \times 10^{-8}\ q/(b\ k_d \tan \beta) \qquad \text{(Equation 2.10-5)}$$

with: h (m), q (Ltd), b (m), and $k_d$ (m/s).

For a 2% slope ($\tan \beta = 0.02$) and a hydraulic conductivity of the leak detection system, $k_d = 10^{-2}$ m/s (1 cm/s), Equation 2.10-5 becomes:

$$h = 2.3 \times 10^{-4}\ q/b \qquad \text{(Equation 2.10-6)}$$

with: h (m), q (Ltd), and b (m).

Table 2.10-1, established using Equation 2.10-6, shows that a head of 0.3 m (1 ft) is obtained for a width of the flow of 1.5 m (5 ft) (which may be considered as a reasonable value and a leakage rate of 2000 Ltd (gpad)). The results in Table 2.10-1 illustrate that, in a typical case, the Rapid and extremely Large Leakage (RLL) is on the order of 2,000 Ltd (gpad).

This example also shows that the calculated head is dependent on the width of flow, b. Unfortunately, the correct value for b is unknown and more information will be required to develop guidelines for the selection of this parameter.

### 2.10.4 Sources of Liquids other than Leakage

### 2.10.4.1 Introduction

#### 2.10.4.1.1 Scope

Sources of liquids other than leakage in the LDCRS include:

- rainwater entrapped in the leak detection drainage layer during construction, which will drain progressively by gravity (except for that portion of the water held by capillarity);
- water present in the low-permeability soil component of a top composite liner, which will be expelled when the soil component compresses under pressures exerted by the waste; and
- ground water intruding into the leak detection system through the bottom liner.

The purpose of this section is to evaluate the flow rate that can be generated by these sources of water and to determine if this can significantly disturb leakage rate monitoring.

#### 2.10.4.1.2 Organization of the Section

The three sources of water are discussed successively:

- Entrapped rainwater in Section 2.10.4.2;
- Consolidation water in Section 2.10.4.3; and
- Ground water in Section 2.10.4.4.

### 2.10.4.2 Rainwater Entrapped in the Leak Detection System

In this section the rate of flow of water entrapped in the leak detection drainage layer is estimated. It is assumed that all water initially entrapped in the drainage layer is due to rainfall that occurred during construction.

It is conservatively assumed that during construction the rainwater collected is not removed from the sump, and that this precludes free draining of the rainwater during construction. As a result, all the rainfall during construction is entrapped in the leak detection system. The volume of water thus entrapped is:

$$V = e\,A\,\Delta t \qquad \text{(Equation 2.10-7)}$$

where: $V$ = volume of water entrapped in the leak detection drainage layer; $e$ = rainfall impingement rate; $A$ = considered surface area of leak detection system; and $\Delta t$ = duration of rainfall. Recommended SI units are $V$ ($m^3$), $e$ (m/s), $A$ ($m^2$), and $\Delta t$ (s).

The maximum time it takes the water to reach a collection pipe is given by Darcy's equation:

$$t_d = \frac{n\,L}{k_d \sin \beta} \qquad \text{(Equation 2.10-8)}$$

where: $t_d$ = drainage time (time for the water to reach a collection pipe); $k_d$ = hydraulic conductivity of the leak detection system material drainage layer; $\beta$ = slope of the drainage layer; $n$ = porosity of the drainage medium; and $L$ = the maximum drainage distance to a collection pipe (conservative). Recommended SI units are: $t_d$ (s), $k_d$ (m/s), and $L$ (m).

Assuming a constant flow rate to the leak detection sump, the rate of flow of water collecting at the leak detection sump may be calculated as follows:

$$Q = V/t_d = e\ A\ \Delta t\ k_d\ \sin \beta/(L\ n) \qquad \text{(Equation 2.10-9)}$$

where Q = rate of flow of water collecting at the leak detection sump ($m^3/s$), and V, $t_d$, e, A, $\Delta t$, $k_d$, $\beta$, n and L as defined above.

Table 2.10-2 has been established for the following conditions:

- Rainfall: 0.025 m/week (1 in./week) (i.e., $e = 4.1 \times 10^{-8}$ m/s)
- Rainfall duration: 1 week
- Slope: 2%
- Hydraulic conductivity of leak detection drainage material: $10^{-4}$ to 1 m/s ($10^{-2}$ to 100 cm/s)
- Porosity: 0.3
- Drainage distance: 15 to 60 m (50 to 200 ft).

Table 2.10-2 indicates that the rate of flow of the water to a leak detection sump increases as the permeability of the leak detection drainage material increases. Also, higher rates will drain for a shorter period of time than lower rates.

The rate of flow in the case of a leak detection system material having a hydraulic conductivity of $10^{-2}$ m/s (1 cm/s) appears to be extremely large (10,000 to 100,000 Ltd (gpad)), but the duration of flow is only a few hours.

Therefore, the drainage of rainwater entrapped in the leak detection system during construction should not significantly disturb leakage rate measurements if the hydraulic conductivity of the leak detection system material is equal to or higher than $10^{-2}$ m/s (1 cm/s).

It is important to note that this conclusion has been drawn from calculations performed using Darcy's equation, i.e., calculations related to flow under steady-state conditions. Therefore, the above conclusion means that rainwater entrapped during construction does not affect the leak detection time (i.e., the detection time assuming flow in steady-state condition). It does, however, affect the initial detection time. Rainwater entrapped during construction will increase the degree of saturation ($S_r$) of the LDCRS drainage medium, thereby decreasing capillary stresses and thus the initial detection time.

### 2.10.4.3 Water Expelled by Consolidation from Top Liner

#### 2.10.4.3.1 Introduction

- Presentation of the Mechanism

The low-permeability compacted soil layer of a composite top liner consolidates under the pressure exerted by the solid waste (for landfill) or liquid waste (for surface impoundments). The pressure exerted on the low-permeability compacted soil layer creates a buildup of excess water pressure in the soil pores which will tend to dissipate by drainage into the leak detection system.

Consolidation of the low-permeability compacted soil layer results in settlement of the soil. Primary consolidation refers to the compression of the soil which occurs as a result of pore water pressure dissipation. Secondary consolidation refers to the compression that occurs at constant effective stress as a result of the rearrangement of soil structure, following the completion of primary consolidation. This section addresses only the water expelled by the low-permeability compacted soil layer during primary consolidation.

- Assumptions Regarding Low-Permeability Compacted Soil Layer

The calculations performed below assume that:

- The low-permeability soil has been preconsolidated by compaction equipment during construction; its consolidation behavior is governed by the recompression curve (rather than the virgin curve) when it is subjected to pressure exerted by solid waste (for landfills) or liquid waste (for surface impoundments).

- Consolidation of the low-permeability soil layer is one-dimensional (which is a legitimate assumption in the case of a relatively thin layer).

- Excess pore water pressure drains only downward and the maximum distance of drainage through the low-permeability soil layer is therefore the thickness of the soil layer.

- The low-permeability compacted soil layer is fully saturated and the quantity of water expelled is equal to the volume decrease experienced by the low-permeability compacted soil layer as a result of settlement; this assumption is reasonable because low-permeability soils are typically compacted at or slightly above their optimum moisture content (which is close to the saturation moisture content).

- Assumptions regarding Rate of Water Expulsion during Consolidation

Consolidation is assumed to commence after the unit is completely filled. In reality the consolidation process commences well before the unit is full.

The rate of water expelled from consolidation is assumed to be equal to the total quantity of water expelled divided by the active life of the unit (i.e. time it takes to fill the unit) plus time at which most of the consolidation has taken place.

### 2.10.4.3.2 Analysis

- Total Settlement

The settlement of the low-permeability compacted soil layer may be estimated based on parameters obtained in a consolidation test:

$$s = \frac{C_r}{1 + e_o} H \log (1 + \sigma/\sigma_o) \qquad \text{(Equation 2.10-10)}$$

where: $s$ = total settlement of the low-permeability compacted soil layer; $C_r$ = recompression index; $e_o$ = void ratio; $H$ = thickness of low permeability soil layer; $\sigma$ = vertical pressure exerted on the surface of the low permeability soil layer by the solid waste or liquid waste; $\sigma_o$ = existing pressure in the middle height of the low-permeability compacted soil layer prior to placement of any waste or overlying leachate collection system materials. Recommended SI units are: $s$ (m), $H$ (m), $\sigma$ (N/m²), $\sigma_o$ (N/m²). $C_r$ and $e_o$ are dimensionless.

In the absence of data, the recompression index may be estimated from the compression index of a consolidation test virgin curve [Navfac DM-7.1, 1982]:

$$\frac{1}{10} C_c < C_r < \frac{1}{5} C_c \qquad \text{(Equation 2.10-11)}$$

where $C_r$ = recompression index (dimensionless); and $C_c$ = compression index (dimensionless).

The compression index, in turn, may be estimated from the low-permeability compacted soil liquid limit [Navfac DM-7-1, 1982; Bowles, 1977]:

$$C_c = 0.009 \, (LL - 10\%) \qquad \text{(Equation 2.10-12)}$$

where: $C_c$ = compression index (dimensionless); and $LL$ = liquid limit (%).

The vertical pressure exerted on the low-permeability compacted soil layer by the waste is:

$$\sigma = \gamma h \qquad \text{(Equation 2.10-13)}$$

where: $\sigma$ = vertical pressure; $\gamma$ = total unit weight of waste; and h = height of waste. Recommended SI units are: $\sigma$ ($N/m^2$), $\gamma$ ($N/m^3$), and h (m).

The pressure existing in the middle height of the low-permeability compacted soil layer prior to any placement of waste and overlying leachate collection system materials is :

$$\sigma_0 = \gamma' H/2 \qquad \text{(Equation 2.10-14)}$$

where: $\sigma_0$ = existing pressure; $\gamma'$ = submerged unit weight of the low-permeability compacted soil layer; and H = thickness of low-permeability compacted soil layer. Recommended SI units are: $\sigma_0$ ($N/m^2$), $\gamma'$ ($N/m^3$), and h (m).

- Total Quantity of Expelled Water

Assuming the low-permeability compacted soil layer is fully saturated, the volume of water expelled from the consolidation of the low-permeability compacted soil layer is:

$$V = sA \qquad \text{(Equation 2.10-15)}$$

where: V = volume of water expelled; s = settlement of the low-permeability compacted soil layer; and A = considered plan area of top liner. Recommended SI units are: V ($m^3$), s (m), and A ($m^2$).

- Drainage Time

Drainage time is assumed to be the sum of the unit active life (which is the time required to apply the load) plus the time at which most of the consolidation water has drained:

$$t = t_1 + t_2 \qquad \text{(Equation 2.10-16)}$$

where: t = drainage time; $t_1$ = active life; and $t_2$ = time at which most of the consolidation has already taken place.

The time $t_2$ may be calculated as follows [Navfac DM-7.1, 1982]:

$$t_2 = \frac{T_V H^2}{c_V} \qquad \text{(Equation 2.10-17)}$$

where: $T_V$ = consolidation time factor; H = distance of drainage through the low-permeability compacted soil layer (i.e., thickness of the low-permeability compacted soil layer); and $c_V$ = coefficient of consolidation. Recommended SI units are: $t_2$ (s), H (m), and $c_V$ ($m^2/s$). $T_V$ is dimensionless and is given in Figure 2.10-3. In order to ensure that most of the consolidation has taken place, a value of $T_V$ of approximately one should be used, according to Figure 2.10-3.

- Rate of Water Expulsion

The rate of expulsion of water due to consolidation is the volume of water expelled divided by the drainage time:

$$Q = V / t \qquad \text{(Equation 2.10-18)}$$

where: Q = rate of water expulsion; V = volume of water expelled; and t = drainage time. Recommended SI units are: Q ($m^3/s$), V ($m^3$), and t (s).

#### 2.10.4.3.3 Review of Results

- Calculations

The following characteristics were considered for the low permeability soil layer: thickness H = 0.60 m (2 ft); unit weight $\gamma'$ = 18.8 kN/$m^3$ (120 pcf); void ratio $e_0$ = 1; and liquid limit LL = 50%. For the waste, a unit weight of 15.7 kN/$m^3$ (100 pcf) was assumed. For a liquid limit of 50%, Equation 2.10-12 yields a compression index of 0.36. This value of the compression index is substituted into Equation 2.10-11 which then yields a recompression index between 0.036 and 0.072.

Several cases pertinent to landfills were considered using the following parameters in addition to those described previously:

- waste height between 3 m (10ft) and 30 m (100 ft);
- recompression index between 0.036 and 0.072; and
- area of 1000 m² (10,750 ft²).

Table 2.10-3 summarizes the total quantities of water expelled from consolidation of the low permeability soil layer. Table 2.10-3 indicates that the total quantities of water varies from 10,000 to 40,000 liters/1000 m² (10,000 to 40,000 gallons/acre).

The time to attain a given percentage of consolidation was calculated by Equation 2.10-17 considering: H = 0.60 m (2 ft); $c_v$ = $4.6 \times 10^{-7}$ m²/s (0.4 ft²/day) (Figure 2.10-2); and $T_v$ = consolidation time factor, as a function of the percentage of consolidation (Figure 2.10-3). Figure 2.10-4 presents a curve showing the percentage of consolidation versus the calculated time to attain it. Figure 2.10-4 indicates that for a 0.60 m (2 ft) thick low permeability soil layer, most of this consolidation (90 to 95 percent consolidation) may be attained after 100 days following the completion of waste placement in the unit.

Considering an active life of 3 years (1095 days) for the unit and a time of 100 days for most of the consolidation to take place, the drainage time is then 1195 days, according to Equation 2.10-16.

Table 2.10-4 presents the rates of water expulsion calculated with Equation 2.10-18. Table 2.10-4 indicates that the rates of water draining through the leak detection drainage layer as a result of low permeability soil layer consolidation range from 8.5 to 34 liters/1000 m²/day (8.5 to 34 gallons/acre/day).

- Conclusions

The rates of water expelled from consolidation of the low-permeability compacted soil layer were estimated for typical landfill scenarios incorporating a composite top liner. The calculations were carried out only for the primary consolidation of the low-permeability compacted soil layer, assuming that the soil layer is fully saturated. The calculations further considered a variety of other parameters judged to be representative of typical low-permeability soils used in composite top liners. The calculated rates of water expelled from consolidation of the low-permeability compacted soil layer ranged between:

- 8.5 liter/1000 $m^2$/day (8.5 gallon/acre/day) for a waste height of 3 m (10 ft) and a recompression index of 0.036; and

- 34 liter/1000 $m^2$/day (34 gallon/acre/day) for a waste height of 30 m (100 ft) and a recompression index of 0.072.

These flow rates are on the order of leakage rates considered for the Action Leakage Rate or more. Therefore, water expelled from a composite top liner as a result of consolidation of the low-permeability compacted soil component of the top liner is expected to significantly disturb leakage rate measurements. Furthermore, this disturbance is expected to affect leakage rate measurements for a long period of time since consolidation is a slow mechanism.

### 2.10.4.4 Leakage Into a Land Disposal Unit Due to Ground Water

Leakage into a land disposal unit will occur if the ground-water table rises above the bottom part of the bottom FML thereby causing an artesian condition with flow directed toward the inside of the land disposal unit. The inward leakage rate can be simply estimated using the bottom half of Table 2.2-16 in Section 2.2, which is related to composite liners.

It appears that inward leakage rates can be:

- on the order of 0.3-10 Ltd (gpad) if the ground-water table is approximately 0.3 m (1 ft) above the top of the bottom liner; and

- on the order of 10-50 Ltd (gpad) if the ground-water table is approximately 3 m (10 ft) above the top of the bottom liner.

As with the rate of water expelled by soil consolidation, the rate of ground water entering the LDCRS can be significant and in the range of the Action Leakage Rate. Therefore, water from this source, if it is present, is expected to significantly disturb leakage rate measurements.

### 2.10.5 Conclusions

This section has presented information on the selection of technical elements of the RAP (2.10.2), the Rapid and extremely Large Leakage (Section 2.10.3), and sources of liquid other than top liner leakage (2.10.4).

### 2.10.5.1 Technical Elements of the RAP

Section 2.10.2 presents a discussion of the minimum technical elements that should be included in the Response Action Plan (RAP). Six major elements were discussed, these being: (1) a general description of the unit; (2) a description of the hazardous constituents contained in the unit; (3) a description of all events that may eventually cause leakage; (4) a discussion of the factors that can effect the amount of leakage entering the LDCRS; (5) a description of the design or operational mechanisms that can be used to prevent the migration of hazardous constituents out of the unit; and (6) an assessment of the effectiveness of a range of possible response actions.

### 2.10.5.2 Rapid and extremely Large Leakage

Table 2.10-1 provides a preliminary assessment of the expected magnitude of the RLL. These results show that leakage rates on the order of 2000 Ltd (gpad) appear to be a reasonable value. However, it should be kept in mind that the evaluation of the RLL is a site-specific determination and highly dependent upon the assumed leak size. As a result, the designer will require guidance for the selection of hole size and the width over which flow occurs.

### 2.10.5.3 Source of Liquids other than Leachate

Section 2.10.3 showed that a significant amount of liquid can enter the LDCRS from sources other than leakage through the top liner. These sources include:

- Rainwater entrapped in the leak detection drainage layer during construction, which will drain progressively by gravity. The amount of water which can be collected will be largely a function of the drainage layer properties, however, the leakage rate of a material with a hydraulic conductivity of $10^{-3}$ m/s ($10^{-1}$ cm/s) will be on the order of 10,000 Ltd (gpad). While this is a large quantity, it will occur over a short period of time (a few days) and should not interfere with the long term monitoring of the LDCRS.

- Water present in the low-permeability soil component of a top composite liner and expelled when this soil layer compresses under pressures exerted by the waste. Typical rates at which the water can be expelled are on the order of 10 to 30 Ltd (gpad) and can occur over several years.

- Ground water intruding into the leak detection system through the bottom liner when the ground-water table rises above the elevation of the bottom FML. The rate of leakage can be on the order of 1 to 50 Ltd (gpad), depending upon the elevation of the ground-water table. This leakage can occur indefinitely if the water table remains high.

Table 2.10-1. Hydraulic heads on the bottom liner calculated using Equation 2.10-6 for large leaks flowing over a width b of the leak detection system. Characteristics of the leak detection system used to establish this table are: hydraulic conductivity, $k_d = 10^{-2}$ m/s (1 cm/s); and slope $\beta = 2\%$.

| | Leakage rate, in Ltd (gpad) | | | |
|---|---|---|---|---|
| b | 100 | 1,000 | 2,000 | 10,000 |
| 1.0 m<br>(3.3 ft) | 0.023 m<br>(0.08 ft) | 0.23 m<br>(0.75 ft) | 0.46 m<br>(1.5 ft) | 2.3 m<br>(7.5 ft) |
| 1.5 m<br>(5 ft) | 0.015 m<br>(0.05 ft) | 0.15 m<br>(0.5 ft) | 0.31 m<br>(1 ft) | 1.5 m<br>(5 ft) |
| 2 m<br>(6.6 ft) | 0.010 m<br>(0.03 ft) | 0.10 m<br>(0.3 ft) | 0.21 m<br>(0.7 ft) | 1.1 m<br>(3.6 ft) |
| | Hydraulic head on the bottom liner | | | |

Table 2.10-2. Flow rate in liters/1000m²/day (Ltd) or gallons/acre/day (gpad) [1 Ltd = 1.1 gpad] of water entrapped in a leak detection system as a function of hydraulic conductivity, $k_d$, of the leak detection system material and the drainage distance, L, to a collection pipe.

| | Hydraulic Conductivity, $k_d$ | | | | |
|---|---|---|---|---|---|
| L | $10^{-4}$ m/s ($10^{-2}$ cm/s) | $10^{-3}$ m/s ($10^{-1}$ cm/s) | $10^{-2}$ m/s (1 cm/s) | $10^{-1}$ m/s (10 cm/s) | 1 m/s (100 cm/s) |
| 15 m<br>(50 ft) | 960<br>(26.1) | 9,600<br>(2.61) | 96,000<br>(0.26) | 960,000<br>(0.026) | 9,600.000<br>(0.003) |
| 60 m<br>(200 ft) | 240<br>(105) | 2,400<br>(10.5) | 24,000<br>(1.05) | 240,000<br>(0.105) | 2,400,000<br>(0.020) |

Values of flow rate in Ltd or gpad
(The values in parentheses are those
of drainage time in days)

Table 2.10-3. Total quantities of water expelled from consolidation of low permeability soil layer, q, in liters/1000m² or gallons/acre, as a function of the waste height, h, and the recompression index, $C_r$.

| | Waste height, h | |
|---|---|---|
| $C_r$ | 3 m<br>(10 ft) | 30 m<br>(100 ft) |
| 0.036 | 10,000 | 20,000 |
| 0.072 | 20,000 | 40,000 |

Table 2.10-4. Rates of water expelled from consolidation of low permeability soil layer, q, in liters/1000m²/day or gallons/acre/day, as a function of waste height, h, and the recompression index, $C_r$.

| $C_r$ | Waste height, h<br>3 m<br>(10 ft) | Waste height, h<br>30 m<br>(100 ft) |
|---|---|---|
| 0.036 | 8.5 | 17 |
| 0.072 | 17 | 34 |

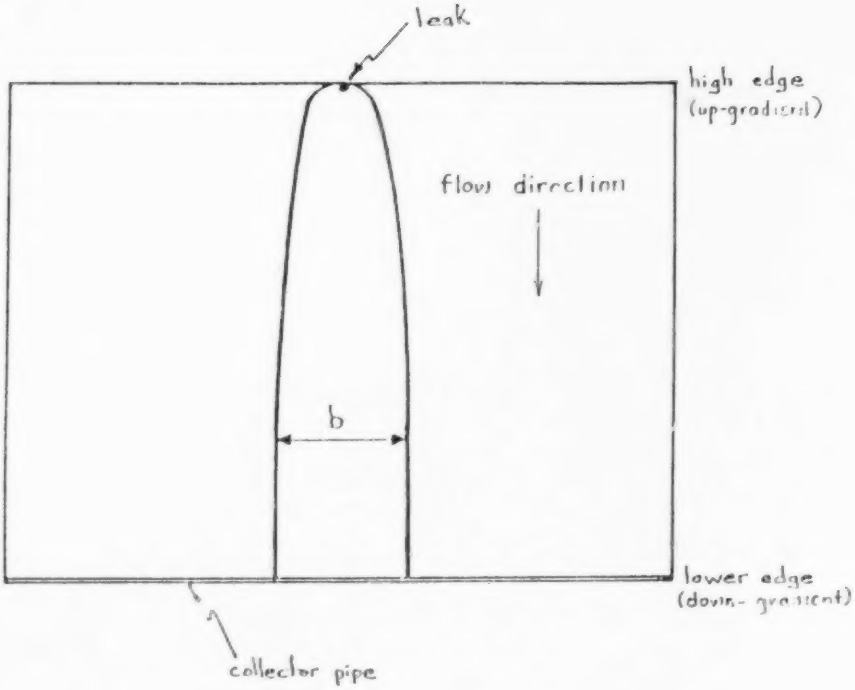


Figure 2.10-1. Plan view of a leak detection system with a large leak flowing over a width b.

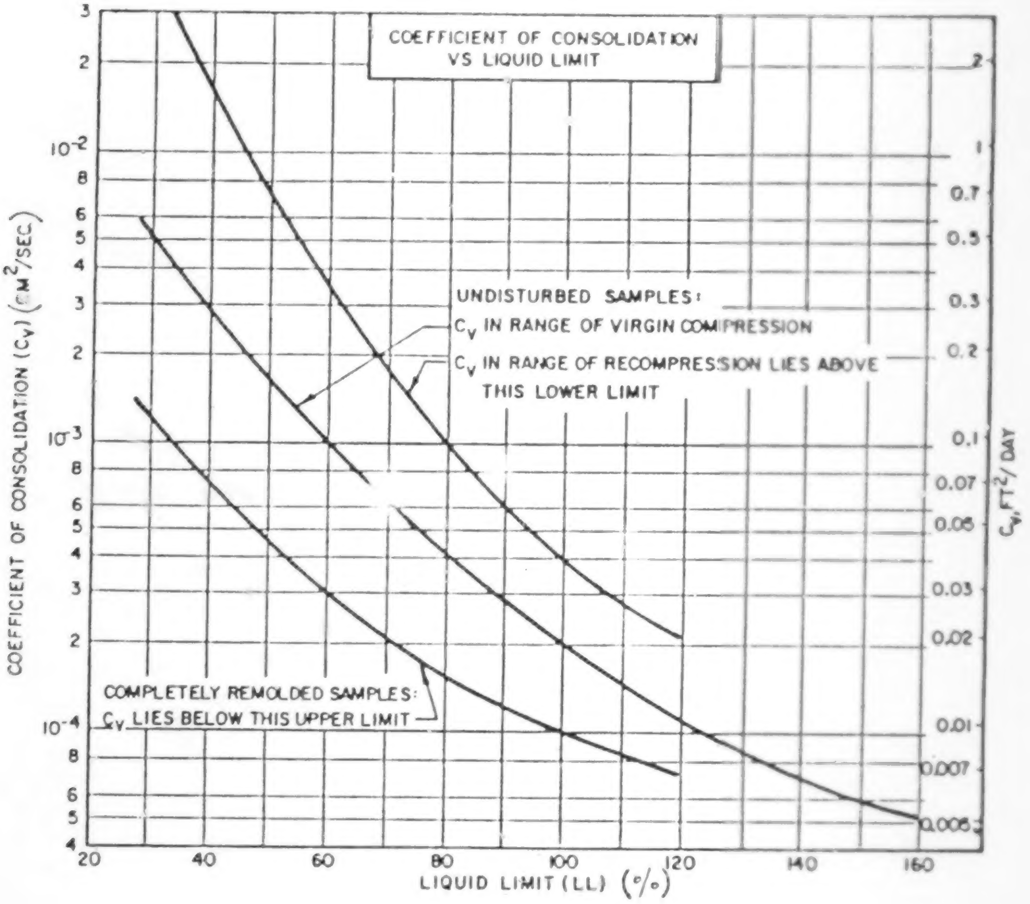


Figure 2.10-2. Coefficient of consolidation, $c_v$, as a function of soil liquid limit, LL. [NAVFAC, 1982]

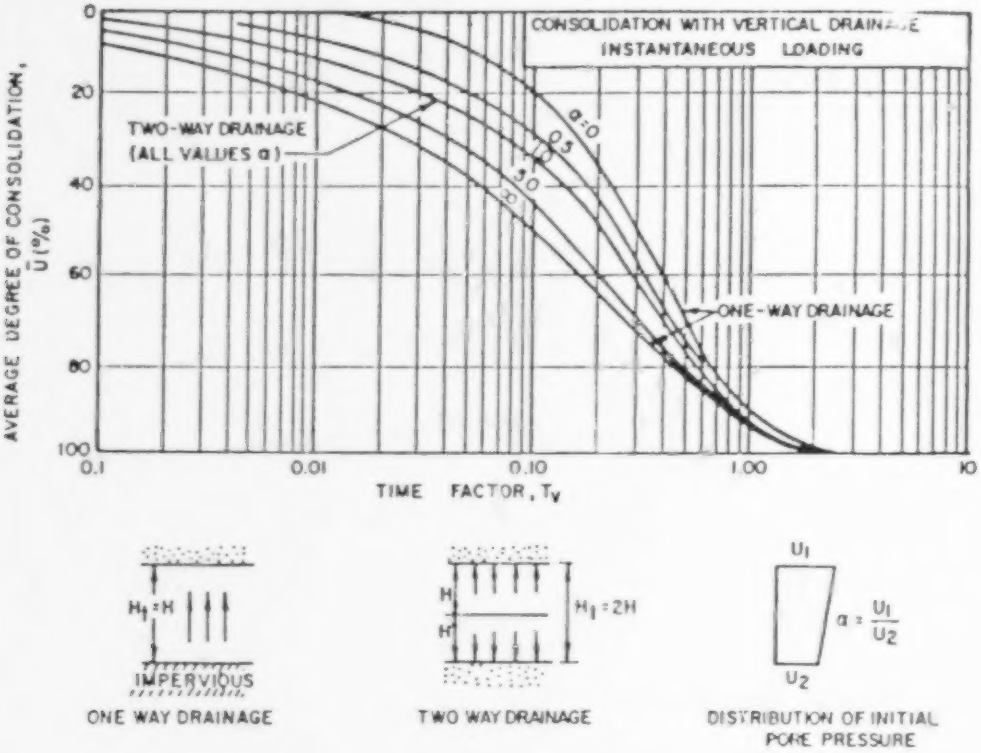


Figure 2.10-3. Consolidation time factor, $T_v$. [NAVFAC, 1982]

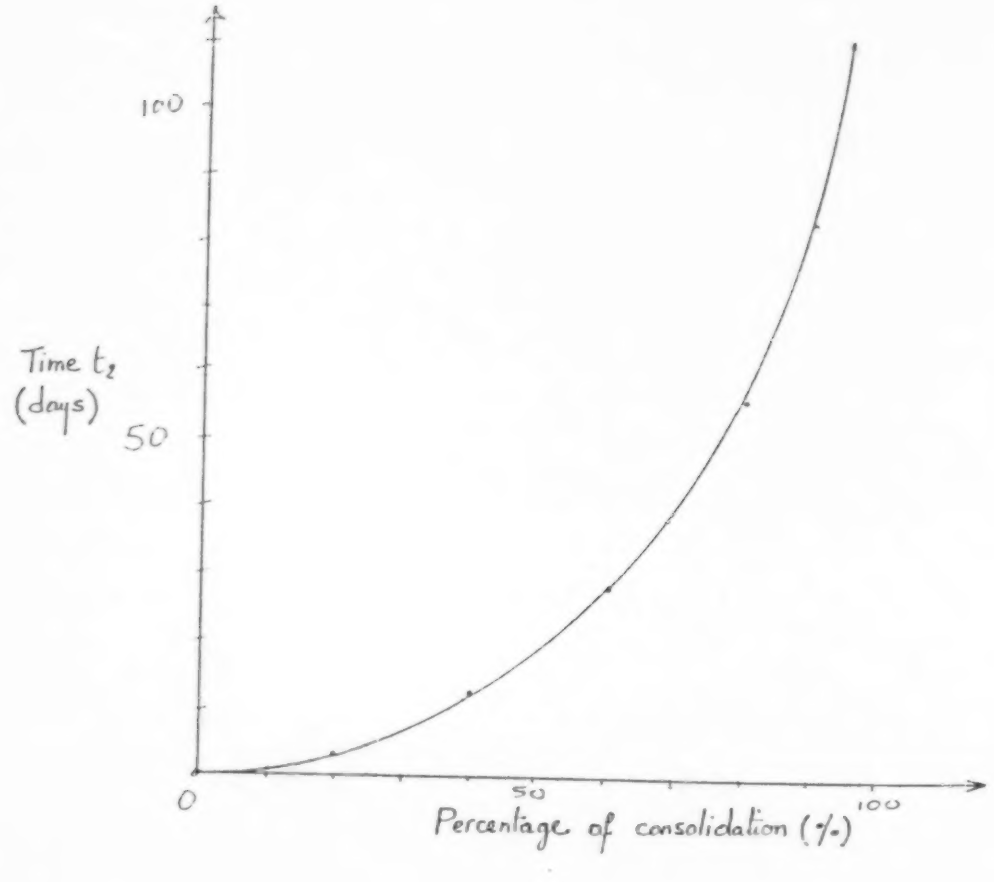


Figure 2.10-4. Time, $t_2$, required to achieve various percentages of consolidation.

# CHAPTER 3

# EXTENSION OF DOUBLE LINER SYSTEM REQUIREMENTS

## 3.1 INTRODUCTION

### 3.1.1 Scope of Chapter 3

The purpose of Chapter 3 is to provide a discussion of the proposed extensions of the minimum technology double liner system requirements to waste piles, significant unused portions of existing facilities, and certain units permitted prior to November 8, 1984. This chapter provides EPA with a summary of the proposed changes to Part 264 and Part 265 regulations, technical rationale for the proposed changes, and regulatory issues and options.

#### 3.1.1.1 Double Liners and LCRS for Waste Piles

EPA is proposing that six months after promulgation (the "effective date") of the proposed Liner/Leak Detection Rule, owners and operators must install double liners and leachate collection systems at new waste pile units, lateral expansions, and replacement units at both permitted and interim status facilities. As a result of this proposed rule, the lining systems at designated waste piles will have technological requirements equivalent to those of landfills and surface impoundments. EPA believes that it is critical that waste piles have equivalent lining systems because, as will be shown in this chapter, the potential for leachate migration from a waste pile can be similar to or greater than the potential for migration from a landfill for an equivalent time period.

Under the proposed rule, owners or operators of permitted and interim status waste piles will be allowed to seek the same variances as those allowed to owners and operators of landfills and surface impoundments from the minimum technology requirements described under Sections 3004(o) (2) and 3004(o) (3) of RCRA. Owners or operators of totally enclosed waste piles that meet the requirements of Section 264.250(c) will remain exempt from these requirements.

### 3.1.1.2 Double Liners and LCRS for Significant Unused Portions of Existing Landfills, Surface Impoundments, and Waste Piles

In the proposed Liner/Leak Detection Rule, existing landfill, surface impoundment, and waste pile units will be required to install double liners and a LCRS between the liners on those portions of the unit that are not defined as existing portions in Section 260.10, do not have a liner system that meets the Part 264 single liner standard, and meet the definition of a significant portion (which is defined subsequently). EPA takes the position that double liners should be installed at significant portions of existing units where the opportunity to do so is the same as for new units. The installation of double liner systems at significant portions of existing units reduces the potential for adverse human health and environmental impacts by preventing, to the extent feasible with current technology, the migration of hazardous constituents out of the unit.

### 3.1.1.3 Double Liners and LCRS for Certain Land Disposal Units at Facilities Permitted Before November 8, 1984

New units, and lateral expansions and replacements of existing landfill, surface impoundment, and waste pile units at facilities permitted before November 8, 1984, will be required to have double liners and leachate collection and removal systems meeting EPA minimum technology standards. This requirement of the proposed Liner/Leak Detection Rule will apply to those units that begin construction 24 months after the date the final rule is published in the Federal Register. The proposed rule, though not required by RCRA, is presented because the potential for migration of hazardous constituents from these units is similar to the potential at units permitted after November 8, 1984. Because units permitted after November 8, 1984 are required to have double liners and leachate collection systems, the Agency believes it is appropriate to require units that are not yet constructed at facilities permitted before November 1984 to also meet these requirements.

There is, however, an exception to the applicability of the requirements discussed above. Under 40 CFR 264.221(f) and 264.254(f), the Agency is proposing to exempt certain replacement surface impoundments and waste piles from the double liner and leachate collection system requirements. In essence, owners or operators who demonstrate that they have a single liner at a surface impoundment or waste pile that currently meets the Part 265 single liner requirements and who have no reason to suspect that the liner is leaking will be exempt from the double liner and leachate collection system requirements.

EPA takes the position that if the owner or operator made a good faith effort to satisfy single liner requirements in effect at the time of permitting, it is unreasonable to require the owner or operator to assume the expenses of a new double liner system.

### 3.1.2 Organization of Chapter 3

Chapter 3 is comprised of four sections which are briefly summarized below.

Section 3.1 is primarily devoted to a discussion of the scope of the proposed extensions to the minimum technology double liner system requirements.

Section 3.2 is concerned with the extension of the minimum technology double liner system requirements to waste piles. This section addresses: description of waste piles; background and issues; comparative performance of waste piles, landfills and surface impoundments; and rationale for proposing double liner system standards.

Section 3.3 is concerned with extension of the minimum technology double liner system requirements to significant portions of existing facilities. This section addresses: definition of significant portions; background and issues; performance of lining systems under significant portions; and rationale for proposing double liner system standards.

Section 3.4 is concerned with extension of the minimum technology double liner system standards to new units, replacements and lateral expansions at facilities permitted prior to November 8, 1984. This section addresses: scope of proposed rule changes background and issues; and rationale for proposing double liner system standards.

## 3.2 WASTE PILES

### 3.2.1 Description of Waste Piles

Waste piles are defined as facilities that store or treat waste in piles (40 CFR 264.250). A waste pile is typically, but not necessarily, an above-ground facility. A typical waste pile is illustrated in Figure 3-1. A prerequisite for waste piles is that at the end of the active life of the waste pile, all waste residues are either removed or decontaminated, as are all contaminated containment system components, contaminated subsoils, and structures and equipment contaminated with waste or leachate. If all wastes, waste residues, and contaminated materials cannot be removed or decontaminated at the end of the active life, the facility must be closed in accordance with closure and post-closure care requirements that apply to landfills.

EPA has previously estimated the number of existing waste piles containing hazardous waste at about 80 units. Of the 80, it is believed that 8 were permitted and 72 are operating under interim status. The exact number of hazardous waste piles currently permitted or operating under interim status is not documented. The current number is believed to be somewhat larger then 80 units. These waste pile units are used in a variety of industrial applications and for a variety of purposes. These include temporary storage of hazardous waste, ore storage at mining facilities, and heap leach pads.

While the operating characteristics of waste piles varies from waste pile to waste pile, some generalized characteristics may be defined:

- waste piles have long active lives (the active life of a waste pile may be as long as the active life plus post-closure care period of a landfill);
- waste piles are usually not covered;
- waste piles are frequently active facilities; and
- lining systems underlying waste piles are more prone to damage from heavy equipment use than are lining systems at landfills.

### 3.2.2 Background

40 CFR 264.251(a) currently requires permitted waste piles to have a single liner that is designed, constructed and installed to prevent any migration of leachate out of the waste pile and into the surrounding environment during the active life (and the closure period, if applicable) of the waste pile. The liner may be constructed of materials (such as low-permeability soils) that allow migration of leachate into the liner itself as long as the leachate does not migrate into adjacent soil, ground water, or surface water. A leachate collection and removal system that is designed, constructed, maintained, and operated to collect and remove leachate from the waste pile is required to be placed immediately above the liner (40 CFR 264.251(a)). Owners or operators whose waste pile is inside or under a structure that provides protection from precipitation, so that neither run-off or leachate is generated, are exempted from liner and leachate collection and removal system requirements provided that: (1) liquids or materials containing free liquids are not place in the waste pile; (2) the waste pile is protected from surface water run-on by the structure or in some other manner; (3) the waste pile is designed and operated to control the dispersal of waste by wind, where necessary, by means other than wetting; and, (4) the waste pile will not generate leachate through decomposition or other actions. For interim status units, with respect to waste received after May 8, 1985, the owner or operator of a waste pile is subject to the requirements for liners and leachate collection systems under 40 CFR 264.251 with respect to each new unit, replacement of existing unit, or lateral expansion of an existing unit that is within the area identified in the Part A permit application. The variance provisions under 264.251 are applicable to these units.

EPA is proposing to modify the current regulations to require double liners and leachate collection and removal systems at new waste pile units, lateral expansions, and replacement units. This action is being proposed based on evidence (discussed subsequently) which indicates that waste piles pose a potential threat to human health and the environment similar to that posed by landfills.

### 3.2.3 Rationale for Double Liner System Requirements

EPA's rationale for a double liner system requirement at waste piles is based on the premise that landfills and waste piles pose similar potential risks to human health and the environment and should be regulated in such a way as to provide the public and environment with equal levels of protection. The premise has been derived from: (1) comparison of typical operating characteristics of landfills and waste piles; (2) analytical calculations comparing the lining system currently allowed in 40 CFR Part 264 with the proposed minimum technology double liner system; and (3) comparison of numerical simulations of leachate generation and migration in landfills and waste piles.

#### 3.2.3.1 Operating Characteristics

Certain operating characteristics of waste piles may result in conditions that increase the potential for leachate migration from the waste pile unit beyond the levels of migration observed in landfills.

For instance, at some waste piles, the waste is periodically removed and replaced with new waste. This moving process involves the use of heavy equipment. The equipment has the potential to damage the lining system through careless operation, construction accidents, or insufficient protective cover above the lining system. Thick protective covers can, to some degree, mitigate the potential for equipment related damage. However, in a typical facility, the thickness of the protective cover will be limited to 0.3 m (1 ft.) to 0.6 m (2 ft.) and the potential for equipment related damage will be present. In contrast, at landfills, waste is not removed above the liner, and the liner is not exposed to equipment operation to the same extent as in waste piles. Thus, the potential for equipment related liner damage appears to be greater for waste piles than for landfills. This potential cause of liner damage increases the probability that hazardous constituents could migrate out of the waste pile.

Moreover, there are other factors which tend to indicate that waste piles possess a potential for leachate generation and migration equaling or exceeding the potential for migration from landfills. Waste piles generally have a higher percentage of their waste area exposed to precipitation than do landfills. In addition, waste is generally exposed to precipitation for a longer period of time at waste piles than at landfills. This is because landfill units are periodically closed by placing a temporary or final cover over the in-place waste. The normal operating practice for landfills is to minimize the time that waste is exposed to precipitation in order to to minimize the generation of leachate. Also, as wastes are removed from an unprotected waste pile and replaced with new waste, more hazardous constituents become available for exposure to precipitation, and therefore, more constituents are available for leaching from precipitation in a waste pile than in a landfill. In addition, the active life for a new landfill unit is typically 6 months to 5 years, while waste piles may be used for storage for a much longer time period, in some cases 20 years or more.

Because the potential for liquids to migrate through a liner is similar for landfills and unprotected waste piles, the same level of protection of human health and environment should be provided at landfills and unprotected waste piles. This equates to installing a minimum technology double liner system at waste piles.

In general, waste piles are also equivalent to landfills with respect to the types of hazardous constituents that will be accepted by the owners and operators of these units. This fact can be seen in the results of the 1986 "National Survey of Hazardous Waste Treatment, Storage, Disposal and Recycling Facilities", conducted by EPA in 1986 [USEPA, 1986d]. Data from this survey are shown in Table 3.1. Presented in the table are the number of landfill and waste pile units found by the survey to "treat, store, dispose, or recycle" each of eight different classification of hazardous waste. While we were not able to determine the total number of landfill and waste pile units included in the survey, we have made estimates of these numbers in order to compare the percentages of landfill and waste pile units accepting hazardous waste from each of the eight considered waste categories. The estimated percentages are also shown in Table 3.1. From inspection, it can be seen that similar percentages of landfills and waste piles accept each of the eight considered waste types. On this basis, the conclusion is drawn that the owners and operators of landfills and waste piles will, in general (there are, of course, exceptions), accept similar types of hazardous waste at their landfill and waste pile units. Therefore, with respect to the waste type contained at the unit, landfills and waste piles can be considered to be roughly equivalent.

### 3.2.3.2 Analytical Calculations

Current 40 CFR Part 264 regulations for waste piles require a single liner "that is designed, constructed and installed to prevent any migration of wastes out of the pile into the adjacent subsurface soil or ground water or surface water at any time during the active life (including the closure period of the waste pile)". The regulations also require a "leachate collection and removal system that is designed, constructed, maintained, and operated to collect and remove leachate from the waste pile. The Regional Administrator will specify design and operating conditions in the permit to ensure that the leachate depth over the liner does not exceed 30 cm (one foot)".

The single liner requirement described above can be satisfied using a compacted soil liner. The performance of this compacted soil liner can be evaluated in terms of the potential for migration of leachate into the bottom-most liner in the lining system as described in the minimum technology double liner system in the proposed Double Liner Rule of March 28, 1986 (51 FR 10706-10723). For leachate to migrate out of a lining system and into the environment, it must pass through the bottom-most liner in the lining system. Thus, an evaluation of migration into the bottom-most liner provides an indication of the potential for migration out of the unit. The performance of compacted soil liners is evaluated below using a one-dimensional, steady-state saturated flow analyses based on Darcy's Equation (Equation 2.4-2). Theses results are then compared to the results on leakage into and through FML and composite liners.

Using Darcy's Equation, the steady-state flow into a uniform layer of saturated compacted soil can be calculated. If it is assumed that the compacted soil has a hydraulic conductivity, $k_c$, of 1 x $10^{-9}$ m/s (1 x $10^{-7}$ cm/s) the following results will be obtained for leakage into (and through) the liner (given in units of Ltd (gpad)):

| Compacted soil thickness, m (ft) | Hydraulic head on liner | |
|---|---|---|
| | h=0.03 m (0.1 ft) | h=0.3 m (1 ft) |
| 0.9 (3 ft) | 89 | 112 |
| 1.5 (5 ft) | 88 | 103 |
| 3.0 (10 ft) | 87 | 95 |

From the above table it can be seen that the steady-state leakage into and through a compacted soil liner is at least 89 Ltd (gpad) for $k_c = 1 \times 10^{-9}$ m/s ($1 \times 10^{-7}$ cm/s) and a liner thickness of at least 0.9 m (3 ft). The amount of leakage into and through a compacted soil liner will be larger if the liner is thinner than 0.9 m (3 ft) or more permeable than $k_c = 1 \times 10^{-9}$ m/s ($1 \times 10^{-7}$ cm/s). From this simple result, it can be concluded that for waste piles generating leachate in excess of 89 Ltd (gpad), at least 89 Ltd (gpad) will migrate into the compacted soil liner.

The value of leakage into a compacted soil liner given above can be compared to a minimum technology double liner system comprised of a FML top liner and a composite bottom liner. (Note: A composite bottom liner has been selected based on EPA's April 1987 Background Document on "Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments" [USEPA, 1987] which showed that composite bottom liners provide higher levels of leachate containment and improved LDCRS leak detection sensitivities and leachate collection efficiencies than compacted soil bottom liners.) Using the results from Section 2.2 of this document, a properly designed top FML installed with good construction quality assurance monitoring may have on the order of one FML hole per acre. Depending on the hole size, the leakage through the hole may be in the range of 30 to 300 Ltd (gpad) for a hydraulic head on the top liner of 0.03 m (0.1 ft), as shown in Table 2.2-16. Leakage through the top liner will be greatly reduced if the top liner is a composite rather than a FML alone. Furthermore, most of this liquid will be collected by the LDCRS. (A small amount may migrate into the composite bottom liner.) Using

Table 2.2-16 and a hydraulic head of 0.03 m (0.1 ft) on the bottom liner (which is very conservative) and assuming one "standard" hole per acre (with good contact), the estimated leakage into the bottom liner should not exceed 0.02 gpad.

By comparing the above results of a single compacted soil liner and a minimum technology double liner system, it can be observed that leakage into the bottom liner (and potentially out of the unit) is on the order of 5000 times more for the single compacted soil liner than for the minimum technology double liner system. (Note: The previous calculation treated the double liner system very conservatively. A less conservative calculation might show that the double liner system reduces the leakage into a composite bottom liner by a factor of several tens of thousands or more compared to the leakage into a single compacted soil liner.)

An important question to consider in evaluating the equivalence of landfills and waste piles is the leachate production potentials of the two units. In general, it is believed that waste piles have longer active lives (10 to 20 years) than landfills (1 to 5 years). Therefore, waste piles will be open to proportionally more precipitation than landfills (which are covered at the end of their active lives). To illustrate this point the following scenario is considered: a 4,000 $m^2$ (1 acre) landfill and a similar sized waste pile are located in an area receiving 0.4 m (15 in.) of rainfall annually. The active life of the landfill is 2.5 years, while the active life of the waste pile is 15 years. It is assumed that in both units only 25% of the precipitation impinging the unit reaches the LCRS above the top liner, with the rest going into surface-water collectors, evaporation and field storage (25% may be realistic for landfills and is probably conservative for waste piles). Based on the above assumptions, at the end of its active life the LCRS in the landfill will have intercepted 1,000 $m^3$ (250,000 gallons) of leachate. In contrast, at the end of its active life, the LCRS in the waste pile will have intercepted 6,000 $m^3$ (1,500,000 gallons) of leachate. From this very simple comparison, it is clear that a waste pile at a given site has a leachate production potential at least equal to, and probably greater than, the leachate production potential of a landfill at the same site.

### 3.2.3.3 Numerical Simulations

In 1983, EPA Office of Solid Waste conducted a study entitled "Evaluation of Land Disposal Facility Technologies and Integration of Waste/Environment/Technology Characteristics to Produce Facility Profiles" [EPA, 1983b]. In this study, EPA evaluated the hydrologic performance of landfills, surface impoundments, and waste piles using the Hydrologic Evaluation of Landfill Performance (HELP) computer program [Schroeder, P.R., et al., 1984a, 1984b]. The HELP program is a water-budget model that can be used to estimate the magnitudes of various components of a water budget at a land disposal unit. For instance, the HELP model can be used to estimate the volume of leachate generated at any point in time at the base of a landfill or waste pile. The model can also be used to estimate the migration of leachate out of a land disposal unit. However, to carry out this last calculation, simplifying assumptions are required regarding migration through the lining system. Results from the HELP model can be used to compare the leachate production potential of a range of simulated landfill and waste pile facilities. Input to the HELP model includes climatologic, unit (liners and leachate collection and removal layers), soil, and waste data. The output from the model can include daily estimates of water and leachate movement into, through and out of the land disposal unit.

EPA used the HELP model to evaluate leachate migration out of landfills, surface impoundments, and waste piles at three different geographic locations (Hartford, CT, New Orleans, LA, and Denver, CO). The rate of migration out of the units was investigated for a 100 year unit life, which included each unit's active life, post-closure care period, and post-care period. The rate of migration out of each unit was investigated for all three types of units as a function of various types of liner and cover systems (using both clays and FMLs).

Results from the study showed that the migration of leachate out of a unit was controlled largely by the assumptions regarding the performance of the FML components of the lining and cover systems. If the FML was assumed to be intact and functioning properly, migration out of the unit was zero. If, however, the FML was assumed to fail at some point in time (e.g., 50 years), the migration of leachate out of the unit would suddenly increase. It is clear that the results presented are dependent on the assumptions about the performance of the FML components of the lining and cover systems. Once the FML components failed and migration out of the unit was initiated, the rate of migration was largely controlled by climatic conditions (i.e., the rate was proportional to the average annual rainfall), and by the permeability of clay layers in the cover and lining systems.

While the absolute results from the EPA study should be used with caution (since the assumptions about long-term FML liner performance are, by necessity, somewhat arbitrary), they are useful for direct comparisons of landfill and waste pile units having similar lining systems and operating characteristics, and exposed to similar climatologic conditions. The results from these types of comparisons showed that for a similar set of climatologic and lining system assumptions, migration rates of liquid into the lining system are similar for landfills and waste piles. Therefore, for a given set of climatologic and unit operating conditions, equivalent lining systems would be required at landfills and waste piles in order to provide protection to human health and the environment.

### 3.2.4 Exemption for Totally Enclosed Waste Piles

Some waste piles are completely enclosed within structures and are thereby protected from wind, rain and snow. If the waste placed in an enclosed facility is dry the waste pile will not generate leachate during its active life (assuming that the enclosure structure continues to function as planned). Since the potential for migration of leachate from the waste pile is non-existent, EPA is proposing to exempt these units from minimum technology double liner system requirements. This exemption is consistent with the existing EPA exemption for enclosed waste pile (40 CFR 264.250(c)).

For a waste pile unit to be eligible for an exemption under 40 CFR 264.250(c), the unit must be totally enclosed and it must be shown that there is no potential for the migration of leachate or hazardous constituents into the surrounding environment. If an owner or operator of a waste pile desires to be exempt from the double liner and leachate collection and removal system requirements, the following conditions under Section 264.250(c) must be met: (1) the waste pile must be inside or under a structure that provides protection from precipitation so that neither run-off or leachate is generated; (2) liquids or materials containing free liquids may not be placed in the pile; (3) the pile is protected from surface water run-on by the structure or in some other manner; (4) the pile is designed and operated to control dispersal of the waste by wind, where necessary, by means other than wetting; and (5) the pile will not generate leachate through decomposition or other reactions. It is important to recognize that the foregoing limitations require that the waste in the waste pile have such a low water content that no free liquids will be present, and further that no leachate will drain out of the waste pile at any time after placement. Therefore, totally enclosed waste piles which contain moist waste or where liquids are added to the waste do not qualify for the exemption. It is recognized that since they are enclosed, waste piles with moist waste have a greatly diminished capacity for leachate generation compared to waste piles exposed to the environment with equally moist wastes. However, since the active life and operating practices (frequency of waste "turnover") of the waste pile are unrestricted, significant amounts of leachate can be generated within enclosed units or liquids can be added to the waste. In addition, in an enclosed waste pile, no restrictions exist on the height of leachate above the top liner other than the existing 40 CFR 264.250(c) requirement to limit the hydraulic head to no more than 30 cm (one foot). This level of liquids above the top liner represents a mechanism for migration potential similar to that for landfills and unenclosed waste piles. EPA believes it is appropriate to require minimum technology double liner systems for enclosed waste piles containing moist wastes that will generate leachate.

EPA's new proposed regulations add an enclosure inspection program to the enclosed waste pile requirements for permitted and interim status units under Section 264.250(c). If the enclosed waste pile does not need a top and bottom liner, then a leak detection system that employs a leachate collection removal system between the liners, as proposed for unprotected waste piles, would not be possible. In recognition of this, EPA will allow the owner or operator of enclosed waste piles to institute an enclosure inspection program as an alternative leak detection system. EPA believes that an enclosure inspection program would satisfy the requirements of Section 3004(o)(4) of RCRA when the owner or operator inspects the enclosure and waste pile after every precipitation event (i.e., rain, snow, or ice) and check for leaks.

### 3.2.5 Variances

Current regulations provide owners or operators of permitted (40 CFR 264) and interim status (40 CFR 265) surface impoundments and landfills with certain exemptions from the minimum technology double liner standards. One type of exemption (e.g., 40 CFR 264.221(d)) applies if the owner or operator can demonstrate that alternative design and operating procedures, together with location characteristics, will prevent the migration of any hazardous constituents into ground water or surface water at least as effectively as a minimum technology double liner system. The second type of variance (e.g. 40 CFR 264.221(e)) applies to certain types of monofills. EPA is proposing to extend to waste piles these two types of exemptions for landfills and surface impoundments. It is EPA's position that extension of these exemptions to waste piles is appropriate because: (1) waste piles falling under the exemptions will handle similar wastes as landfills and surface impoundments; and (2) waste pile lining systems have similar designs and design lives as landfills and surface impoundments.

Owners or operators of permitted and interim status units may receive a variance from the minimum technology double liner requirements if they are able to demonstrate that the alternative design and operating procedures, together with location characteristics, will prevent migration of any hazardous constituents into ground water or surface water at least as effectively as the minimum technology double liner system.

## 3.3 SIGNIFICANT PORTIONS

### 3.3.1 Definition of Significant Portions

"Significant portions" has not previously been defined in RCRA or its amendments, but has evolved in concept from regulations pursuant to RCRA. EPA is currently proposing to define significant portions as: "any unlined area of a unit that has not received waste and, if double lined before receiving waste, would significantly reduce the potential for ground-water and surface-water contamination from the unit".

### 3.3.2 Background

EPA's current regulations require the lining of partial units at permit issuance. Section 264.221(a) (for surface impoundments), Section 264.251(a) (for waste piles), and Section 264.301(a) (for landfills) require the portions of units not covered with waste at permit issuance to install a single liner (with a leachate collection and removal system above the liner in the case of a landfill or waste pile). This means that even if a landfill, surface impoundment or waste pile unit is exempt from the double liner standards, any portion of the unit not covered with waste at permit issuance is still subject to EPA's current single liner standards in Sections 264.221 (a), 264,251(a), and 264,301(a). Hence, in this instance, EPA's current standards have not been superseded by HSWA.

EPA believes the number of units that can be characterized as having "significant portions" is small, probably less than 10. However, the potential adverse impact to human health and the environment of not double-lining the "significant portion" is large.

The potential benefit associated with the use of a minimum technology double-liner system compared to a single compacted soil liner (with a LCRS above the liner in the case of a landfill) can be assessed from the results of calculations presented in Section 3.2.3.2. In that section, it was pointed out that the last step before leachate passes through a lining system and into the environment (where it can potentially adversely affect human health and the environment) is its passage into the bottom-most liner in the lining system. Therefore, migration of leachate into the bottom-most liner should be minimized. In Section 3.3.3.2, it was shown that a single compacted soil liner with $k_C$ = 1 x $10^{-9}$ m/s (1 x $10^{-7}$ cm/s) would allow on the order of 89 Ltd (gpad) of leachate to enter into the liner.

In contrast, the migration of liquid into the bottom composite liner of a double liner system is on the order of 0.2 Ltd (gpad) or less. The difference between 89 Ltd (gpad) and 0.2 Ltd (gpad) represents reduction in the potential for migration out of the unit by a factor of 500. Since the above results are conservative, the actual factor may be several thousand rather than 500. If a 5 acre significant portion is considered and it has a 10 year active life, and if the rate of leachate generation in the unit is in excess of 86 Ltd (gpad), the total quantity of liquid migrating into the bottom-most liner will be on the order of 6 million liters (1.5 million gallons) for a single compacted soil liner, and on the order of 15,000 liters (3,600 gallons), or less for the composite bottom liner of a minimum technology double liner system.

### 3.3.3 Rationale for Double Liner System Requirements

The proposed "significant portions" regulations changes the current single liner requirements for unused portions of existing units to a requirement for a double liner system (for a significant portion) or to a complete waiver of liner requirements (for a nonsignificant portion). The objective of this change is to correct inequities in the existing regulations, which would have owners or operators of some units install liners which would not serve to reduce threats to human health or the environment. However, EPA recognizes the need to retain or enhance this protection in areas where liners would serve to reduce the potential for the migration of hazardous constituents out of the unit.

The primary purpose behind a requirement for minimum technology double liner systems at significant portions is to provide these portions with the same level of protection (by controlling migration of hazardous constituents out of the unit to prevent ground water contamination) of human health and the environment that is provided by other newly constructed land disposal units. By requiring a minimum technology double liner system for significant portions, EPA would be minimizing the total number of land disposal units that can receive hazardous waste and not be as protective of human health and the environment as other units with minimum technology double liner systems.

### 3.3.4 Proposed Exemption from Leak Detection Requirements

The proposed Liner/Leak Detection Rule does not require a leak detection system to be installed at significant portions of existing units. Thus, the leachate collection and removal system between the liners would be exempt from the leak detection system performance requirements outlined in Chapter 2. From a technical viewpoint, this is a reasonable position because: (1) the possibility of leakage from other areas of the unit that could cause a false indication of leakage through the top liner of the significant portion; and (2) potential implementation problems from response actions being required for leakage through the top liner when other portions of the unit may not have any liner. These implementation problems are caused by having different operational requirements for the "existing portions" part and the "significant portions" part of a unit. Without having a consistent requirement for migration out of the whole unit into the subsurface, it would be difficult or impossible to determine if the portion of the unit with more stringent operational controls (the "significant portion") is meeting its requirements. This is because current monitoring techniques would not be able to determine which areas of the unit were leaking. Therefore, EPA would not know whether or not the "significant portion" was in compliance with the double liner standards and any potential response actions. However, owners and operators should still be encouraged to voluntarily comply with the minimum technocal guidelines for double liner systems that are set forth in this document.

### 3.3.5 Examples of Significant Portions

Precise criteria have not been developed for categorizing significant portions and nonsignificant portions. However, the following examples provide guidance on EPA's thinking of what are significant and nonsignificant portions:

- An example of a "significant portion" of an existing landfill unit would be an exposed unlined bottom area of several acres that was not covered by waste. If double liners and a leachate collection and removal system were installed in this area prior to its receiving waste, a significant benefit to human health and the environment would likely result because large amounts of leachate would be collected and removed over a 5 year period.

- An example of a portion of an existing unit that may not be a "significant portion" is the unlined area of a surface impoundment located above the liquid surface level that would be covered with waste if the liquid level were raised.

- In most cases, "significant portions" will be those areas in a unit where the addition of a double liner system will provide hydraulic control of leachate or liquid waste and assure collection and removal.

- "Significant portions" may include both the bottom and sidewalls of existing units.

Examples of significant portions are given in Figure 3-2.

### 3.3.6 Variances

In EPA's proposed rule changes for lining systems for significant portions, owners or operators would be eligible for an alternative technology variance. Owners or operators of significant portions of permitted and interim status units wishing to use designs different from those specified under the minimum technology requirements will be allowed to do so if they are able to demonstrate that the alternative design and operating procedures, together with location characteristics, will prevent the migration of hazardous constituents into ground water or surface water at least as effectively as a minimum technology double liner system.

The proposed rule changes also provide a provision for owners or operators of significant portions of permitted or interim status facilities to seek a waiver of the double liner system requirements for monofills containing only hazardous wastes from foundry furnace emission controls or metal casting molding sands if such wastes do not contain constituents which would render the waste hazardous for reasons other than the EPA toxicity characteristics in Section 261.24, 40 CFR Ch.1. Further requirements to obtain such a waiver were given previously in Section 3.2.5 of this report.

## 3.4 NEW UNITS, REPLACEMENTS AND LATERAL EXPANSIONS AT FACILITIES PERMITTED PRIOR TO NOVEMBER 8, 1984

### 3.4.1 Background

The statutory requirements of RCRA (Section 3004(o)(1)(i)) and current regulations specify minimum technology double liner systems only for new units, lateral expansions and replacements of existing landfills and surface impoundments that receive permits after November 8, 1984. As previously discussed in Section 3.3.2, units permitted prior to November 8, 1984, may currently be required to have single liners (and leachate collection and removal systems above the liner for landfills and waste piles) on unused portions of a unit.

### 3.4.2 Rationale for Double Liner System Requirements

EPA is proposing to extend the minimum technology double liner system standards to new units, replacement units, and lateral expansions of surface impoundments, waste piles and landfills at facilities permitted prior to November 8, 1984. The rationale for this proposal is to assure that these units provide the same level of protections of human health and the environment as is provided at other newly constructed units. The EPA's proposal will minimize the number of units in which waste can be placed that are not as protective of human health and the environment as units having minimum technology double liner systems. EPA is of the belief that the opportunities for construction, and impacts on owners or operators, are similar to those for units permitted after November 8, 1984. In other words, the technical and resource requirements for implementing the minimum technology double liner system standards are identical for new units replacements and lateral expansions permitted prior to, and after, November 8, 1984. No additional requirements are placed on owners and operators of units permitted prior to November 8, 1984 that aren't placed on units permitted after that date.

### 3.4.3 Exemptions for Certain Replacement Units

EPA is proposing to exempt certain replacement units permitted prior to November 8, 1984 from the minimum technology double liner and leachate collection and removal requirements, as well as the leak detection system requirements proposed today. (EPA can exempt these units from the leak detection requirements because they are not required by the statute to have leak detection.) The number of units affected by this proposed extension of the double liner standard is small. The total number of affected facilities is believed to be approximately eight. It is believed that all of these cases will involve lateral expansions or replacements (i.e, no new units).

The types of land disposal units that the Agency believes are most likely to be defined as replacement units are surface impoundments and waste piles. As discussed in the Federal Register (50 FR 28742, July 15, 1985), a unit qualifies as a replacement unit when: (a) the unit is taken out of service (i.e., the receipt of waste is stopped or the normal input of waste is significantly reduced); (b) all or substantially all of the waste is removed; and, (c) the unit is reused. However, a unit is not considered a replacement unit if the waste is removed from the unit for treatment, treated, and then put back into the same unit as part of the unit closure plan.

EPA is considering exemptions from the double liner system and leak detection system requirements for those replacements of landfills, surface impoundments, and waste piles that meet the following conditions:

- The existing unit received a final permit prior to November 8, 1984;

- The existing unit was constructed in compliance with the single liner requirements (and leachate collection and removal system requirements for landfills and waste piles) or requirements for equivalent protection (i.e., the variance) contained in Part 264 and the liner or leachate collection and removal system is not replaced; and

- There is no reason to believe that the liner or leachate collection system is not functioning as designed.

EPA is considering exemption of units that meet the above criteria from the double liner system and leak detection system requirements, because the owner or operator of these units made a good faith effort to satisfy the liner system requirements that were in effect at the time the facility was permitted (and the liner or leachate collection system are still functioning as designed); EPA also considered that in many cases the owner or operator would be required to totally replace the whole unit. Retrofitting an additional liner on top of the existing liner would not be feasible. This is for three reasons: (1) existing single liners would not meet double liner system bottom liner requirements; (2) reduced capacity would not meet unit owner or operator needs; and, (3) retrofitting a design concept does not allow the owner or operator to meet new BDAT technology for liners.

### 3.4.4 Variances

In EPA's proposed rule changes owners or operators of new units, replacement units, and lateral expansions of units at facilities permitted prior to November 8, 1984, would be eligible for the same variances as previously described in Section 3.2.5 of this report.

Table 3.1 Comparison of waste types accepted at landfills and waste pile units. [This data is from the EPA 1986 "National Screening Survey of Hazardous Waste Treatment, Storage, and Disposal Facilities" (USEPA, 1986d)]

| | Landfills[1] | | Waste Piles[1] | |
|---|---|---|---|---|
| | No. of Units | (%)[2] | No. of Units | (%)[2] |
| Acidic Corrosives (pH<2) | 35 | 23 | 13 | 16 |
| Metals | 94 | 63 | 75 | 94 |
| Cyanides | 37 | 24 | 4 | 5 |
| Solvents | 37 | 24 | 12 | 15 |
| PCBs | 3 | 2 | 11 | 14 |
| Dioxins | 2 | 1 | 1 | 1 |
| Other Halogenated Organics | 36 | 24 | 12 | 15 |
| Other Hazardous Waste | 90 | 60 | 54 | 67 |
| Total Number of Units (approximate) | 150 | | 80 | |

Notes: (1) The number of units refers to the number of units willing to treat, store, dispose or recycle the particular waste type. The (%) refers to the percentage of waste pile units willing to treat, store, dispose, or recycle that waste type.

(2) Percentages are estimates based on the estimated total number of units.

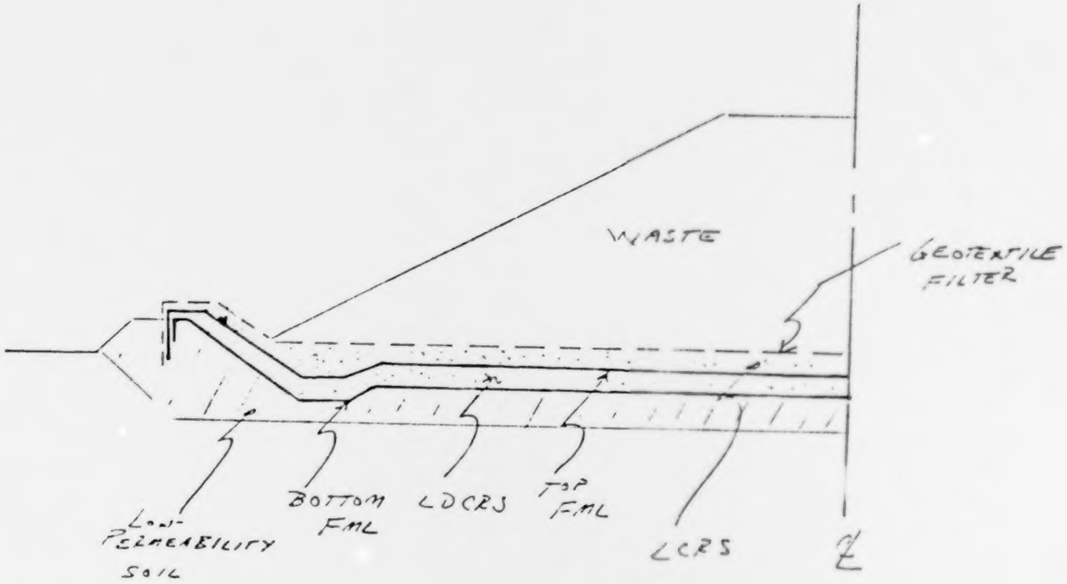
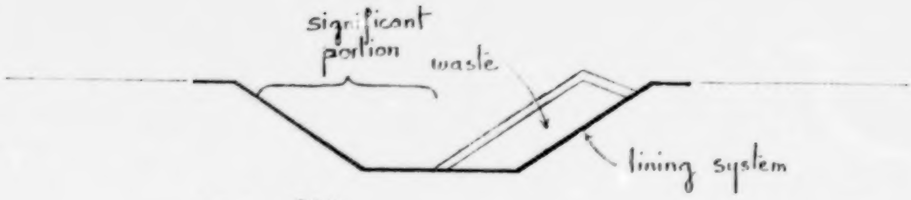
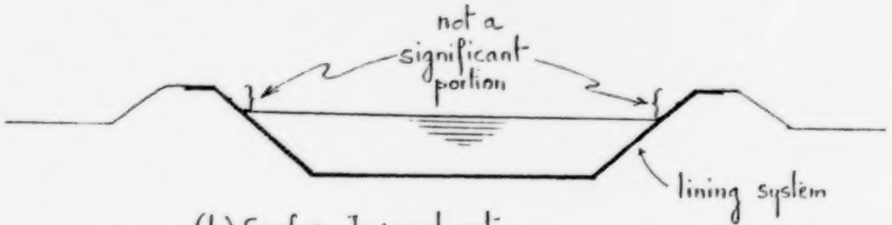


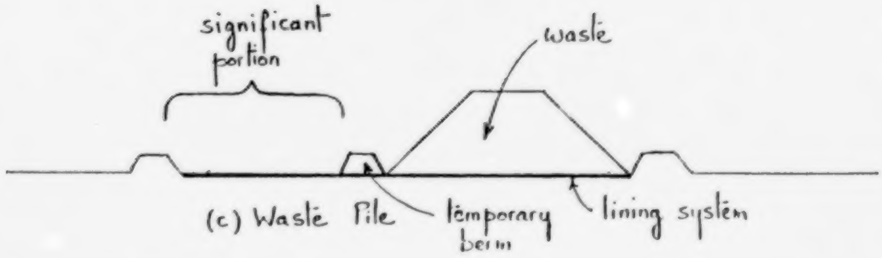
Figure 3-1. Typical waste pile incorporating minimum technology double liner systems.



(a) Landfill



(b) Surface Impoundment



(c) Waste Pile

Figure 3-2. Examples of significant portions.

# CHAPTER 4

# CONSTRUCTION QUALITY ASSURANCE

## 4.1 INTRODUCTION

### 4.1.1 Scope of Chapter 4

The purpose of Chapter 4 is to provide a discussion of Construction Quality Assurance Program requirements in the proposed Liner/Leak Detection Rule. This chapter therefore includes: a summary of the proposed additions to 40 CFR Part 264 and Part 265 Regulations; technical rationale for the proposed changes; and, regulatory issues and options.

Chapter 4 is comprised of 6 sections which are briefly summarized below.

Section 4.1 describes the background and rationale for the Construction Quality Assurance Program, as well as the roles and responsibilities associated with implementation of a CQA Program.

Section 4.2 is concerned with the elements of the written CQA Plan: the organizations which will be involved in the program; the CQA Officer and the CQA personnel; sampling strategies; and reports and documentation.

Section 4.3 addresses the role that the Construction Quality Assurance plays in double liner systems. This section discusses: the sensitivity of system performance to construction procedures; materials issues; and the benefits of Construction Quality Assurance.

Section 4.4 is concerned with the scope of a Construction Quality Assurance Program. This section addresses: the role of CQA during the pre-construction and design stage; CQA tasks during construction; and post-construction CQA, including reporting and monitoring requirements.

Section 4.5 is concerned with the testing procedures used in a Construction Quality Assurance Program. This section addresses: laboratory and field soils tests; soils tests acceptance criteria; laboratory and field flexible membrane liner (FML) tests; FML acceptance criteria; and testing procedures and acceptance criteria for other geosynthetic materials.

Section 4.6 discusses Construction Quality Assurance issues. This section addresses: the qualifications of the CQA Officer; levels of control within the CQA Program; administrative timing of review and approval of CQA Plans; interim status vs. permitted CQA Plan requirements; and length of comment periods.

### 4.1.2 Rationale for the CQA Program

Historically, a conscientious and well-managed Construction Quality Assurance Program has served to ensure that a completed project meets or exceeds the specified design. The provision of quality assurance is a standard component of virtually any project involving the placement and compaction of soils. Similarly, geosynthetics require close monitoring during placement in order to ensure a quality installation. As a result, EPA believes that the Construction Quality Assurance (CQA) Program represents an essential element of its overall liquids management strategy. The CQA Program must ensure that all foundations, low-permeability compacted soils, flexible membrane liners (FML's), dikes, leachate collection and removal systems (LCRS's), and final cover, meet or exceed all design criteria, plans, and specifications.

The first element of the CQA Program is the preparation of a site-specific Construction Quality Assurance Plan. The CQA Plan addresses activities such as monitoring, documenting, and sampling for the individual components. By providing the CQA Plan during the design stages of the unit, permit granting agencies and regulators are able to review the specific procedures that the Owner/Operator will use to comply with CQA requirements. For permitted units, the CQA Plan must be submitted to, and approved by, the Regional Administrator (RA) before construction will be allowed to begin.

The second element of the CQA Program is the implementation of the CQA Plan by the CQA Officer (i.e., a registered professional engineer.) Thorough and complete documentation of all of the monitoring activities and signature(s) noting the compliance with the Construction Quality Assurance Plan are required in the report submitted by the CQA Officer to the owner/operator and by the owner/operator to the Regulatory Agency. The submission and approval of this report are prerequisites to the granting of permission to receive waste at the unit.

The Construction Quality Assurance Program is directly related to both parts of EPA's liquids management strategy: minimizing the generation of leachate and maximizing leachate removal. To ensure that the waste management system will meet EPA's objectives, all components of the total system must function as designed: top and bottom liners, leachate collection and removal systems above and between the liners, the leak detection system, and the final cover. The CQA Program establishes specific activities that the Owner/Operator must implement to ensure the quality of each component of the system.

Use of systematic CQA programs at waste piles, landfills, and surface impoundments will help ensure that each unit is designed and constructed to the same general standards. This applies to the lining systems and covers of waste piles, landfills, and surface impoundments, and to the covers for land treatment units. The latter involves covers only, because the treatment of wastes in these units involves the application of the waste on the soil surface or into the upper soil layers in order to degrade, transform, or immobilize hazardous constituents present (See Chapter 5).

There are significant benefits to the provision of a CQA Program. In particular, by the provision of a comprehensive, thorough program of Construction Quality Assurance, there will be greater scrutiny and diligence, as well as the establishment of formal protocols for conducting examinations and testing for flaws in materials. Consequently, when flaws are found and corrected there will be a reduced potential for migration of hazardous constituents out of the unit and into the environment will be reduced.

By the provision of a high quality and intensive CQA program, the responsible parties (Owner/Operator, Designer, and Regulators) will have greater confidence that every effort has been made to ensure conformance with the design and specifications, and that the unit will therefore perform as intended. As a natural extension, there will be greater confidence in the integrity of the unit in the eyes of the public.

### 4.1.3 Definitions Related to CQA

Within the context of the Construction Quality Assurance Program, many of the terms and definitions used may differ from their common usage. The following definitions are those which are used in Chapter 4. A discussion or clarification is also provided in order to avoid any misinterpretation of these terms.

- Quality Assurance - "means all those planned and systematic actions needed to provide adequate confidence that products or services will satisfy specific requirements." [Canadian Standards Association (CSA), 1986].

  In this document, Construction Quality Assurance includes the provision of quality assurance services for the manufacture, fabrication, and installation of the geosynthetic components of lining systems, including flexible membrane liners (FML's) (also referred to as geomembranes), geotextiles, geonets, and geogrids; as well as the testing, placement, and compaction of the soils components of land disposal units, including foundations and dikes, compacted low-permeability soil layers and high-permeability granular drainage layers. Construction Quality Assurance actually has pre-construction, construction, and post-construction components.

  In addition, Giroud and Fluet [1986] state that: "In the context of geomembrane-lined facilities: ... Quality assurance refers to means and actions employed by the owner through the quality assurance team to assure conformity of the design, production (i.e., manufacture and fabrication) and installation with the quality assurance plan, as well as with drawings and specifications. 'Third party quality assurance' refers to a quality assurance team which is independent of the designer, manufacturer, fabricator, installer or owner".

- Quality Control - "those actions which provide a means to measure and regulate the characteristics of an item or service to contractual or regulatory requirements." [Giroud and Fluet, 1986].

In addition, Giroud and Fluet [1986] state: "In the context of geomembrane-lined facilities: Quality control refers to those actions taken by the designer, manufacturer, fabricator, and/or installer to ensure that their methods, materials and workmanship are accurate and correct and meet the requirements of regulations, plans and specifications. ... Quality control is provided by each party for its own work (e.g., quality control of the installation is provided by the installer) while quality assurance is provided by a party independent from design, production and installation. Quality control is therefore associated with the offering (selling) of a product or service, whereas quality assurance relates to the acceptance of a product or service".

In the case of the quality assurance of soils placement and compaction, the CQA Officer normally undertakes the nondestructive quality assurance testing, and, since the Earthworks Contractor does no other testing, this also serves as quality control testing. In this case, this testing is required to determine the acceptability of the work, and traditionally, Earthworks Contractors have accepted the results of the testing by the CQA Contractor, but this in no way relieves the Earthworks Contractor of his responsibility for conformance.

- Construction Quality Assurance Plan - A Construction Quality Assurance Plan is a site-specific document which describes the Construction Quality Assurance Program. This CQA Plan is normally prepared by the CQA Contractor or Designer, and includes, at the least:

- Site-Specific Application - the name, location, nature, general description, and type of waste management unit;
- Roles and Responsibilities - identifies all parties to the program, qualifications of the parties and their personnel, and their responsibilities and levels of authority;
- Procedures - outlines or includes all procedures to be followed in the performance of the work, including:
    - test procedures;
    - acceptance/rejection criteria;
    - repair procedures;
    - sampling procedures;
    - documentation procedures; and
    - reporting procedures.
- Documentation Requirements - lists all elements of the required documentation, including the final report, signatures required, and the record drawings.

Specific requirements of the Construction Quality Assurance Plan are discussed in Section 4.2.

- Nondestructive Testing - Nondestructive testing consists of procedures for testing the materials in relation to the specifications, without damaging or otherwise requiring reconstruction or replacement of the tested material. For example, in the context of soils testing, nuclear test procedures determine the in situ density of the soil without disturbance of those materials. In this case, a quantitative result is obtained (e.g., soil density). In the context of nondestructive testing of FML's, vacuum testing (for instance) is carried out without damaging the seams. In this case, a qualitative result is obtained (e.g., confirmation of seam continuity).

- Destructive Testing - Destructive testing consists of procedures for testing the materials in relation to the specifications using procedures that result in destruction of a portion of the installed material. Destructive testing thus requires some form of replacement or repair of the installation. In the context of soils testing, destructive testing would normally consist of the field sampling of the compacted soil materials for laboratory testing, which requires replacement of the material removed. In the context of flexible membrane liner seam testing, samples are cut from the field-formed seams for laboratory testing to failure, to determine the ultimate strength properties of the seam. The sampled location must then be repaired.

- Conformance Testing - Conformance testing is carried out on samples collected from the materials supplied to the site, but before installation. This testing is carried out in the laboratory to verify the properties and record the conformance of these properties with the specifications. Sufficient additional materials must be provided to the site such that the samples removed for conformance testing (at a specified frequency) do not result in a shortage of material to complete the work.

### 4.1.4 Parties to CQA - Roles and Responsibilities

There are many parties involved in the Construction Quality Assurance Program and these interact in different ways. It is important to identify these parties, as well as their roles and responsibilities, in the CQA Program.

> As outlined in Giroud and Fluet [1986]: "The following is a listing of the various parties along with a brief description of their roles and responsibilities:
>
> - Designer - responsible for the design, drawings, plans and specification of the lining system and the supporting soil.
>
> - Civil Engineering Contractor - responsible for the preparation of the supporting soil on which the lining system is to be

installed and for the construction of the concrete structures and the pipe systems to which the lining system is to be connected; may also be the party responsible for placing earth or concrete cover and granular drainage or filter materials, if any.

- Polymer Supplier - produces and delivers raw polymer (typically in the form of flakes or pellets) to the manufacturer.

- Manufacturer - responsible for production of geosynthetics from raw polymer. In the case of geomembranes, produces rolls of a constant width.

- Fabricator - responsible for the fabrication of geosynthetic panels from geosynthetic rolls. (Geomembranes are the most likely materials to require fabrication.)

- Transporter - transports geosynthetic rolls from manufacturer to fabricator or the site and/or geosynthetic panels from fabricator to the site. (Transportation is usually not critical for geosynthetics other than geomembranes.)

- Installer - responsible for field handling, storing, placing, seaming and other site aspects of the geosynthetics; may also be responsible for anchor trenches and all temporary anchoring or loading required to support the lining system during installation.

- Quality Assurance Contractor - party (independent from the designer, manufacturer, fabricator, installer and owner) responsible for observing and documenting activities related to the quality assurance of the geosynthetic lining system. ... The quality assurance contractor is the employer of the quality assurance team.

- Quality Assurance Team - the quality assurance team includes: (i), one quality assurance managing engineer, who is located at the offices of the quality assurance contractor and is responsible for managing the quality assurance team; (ii), a quality assurance manager, who is physically present at the fabrication factory and/or site throughout the lining system fabrication and/or installation, and assigns tasks to the quality assurance monitors and otherwise manages their activities; and (iii), quality assurance monitors who are responsible for documentation of all installation activities which they observe. ...

- Quality Assurance Laboratory - party (independent from the designer, manufacturer, fabricator, installer and owner) responsible for conducting tests on samples of geosynthetics taken from the fabrication factory and/or the site. For large projects, the laboratory may be located on site.
- Owner/Operator - owns and/or is responsible for the lined facility. For quality assurance purposes, the term 'owner' usually applies equally to 'operator', i.e., the party responsible for operating the lined facility.
- Project Manager - the official representative of the owner; i.e., the individual in charge of coordinating field activities.
- Regulator(s) - responsible for enforcing compliance with regulatory statutes and/or codes."

In addition:

- Earthworks Contractor - the party responsible for the placement and compaction of soils for earthworks and the compacted soil liner at the site;
- Soil Supplier - the party responsible for supplying soils to the site;
- Soils CQA Contractor - the party (independent from the designer, manufacturer, fabricator, installer and owner)

> responsible for observing and documenting activities related to the quality assurance of the earthworks and compacted soil components of the lining system... The Soils Quality Assurance Contractor is the employer of the Soils Quality Assurance Team and may also be the Geosynthetics Quality Assurance Contractor.

Within the context of this document, the roles of the CQA Manager and CQA Managing Engineer are fulfilled under the title "CQA Officer", whose function may or may not coincide with the two separate roles described above. Similarly, the Quality assurance laboratory is referred to herein as the "Independent Test Laboratory". In addition, the roles and responsibilities of the CQA Contractor, and hence the CQA Team, will also include the Construction Quality Assurance of the soils and other non-geosynthetic components of the work. In any event, the Construction Quality Assurance Plan should clearly delineate the responsibilities of each party.

## 4.2 CONSTRUCTION QUALITY ASSURANCE PLAN

The unit Owner/Operator must submit a written CQA Plan as part of the permit application. Although the overall content of the CQA Plan will depend on the site-specific conditions for the proposed hazardous waste management unit, at a minimum, several elements should be included in the Plan. These elements of the Construction Quality Assurance Plan are summarized in this section.

- General Description of the Unit--Plans for the design, construction, operation, and closure of the unit should be discussed. The description should identify the construction stages for the components at the unit.

- Responsibility and Authority--The responsibility and authority of organizations and key personnel (by title) involved in permitting, designing, constructing, and quality assuring the hazardous waste management unit should be described in the CQA Plan. The description must assure that the objective of the CQA Program identified in 40 CFR 264.19 will be met.

- CQA Personnel Qualifications--The qualifications of the CQA Officer and supporting CQA personnel should be presented in the CQA Plan in terms of the training and experience necessary to fulfill their identified responsibilities.

- CQA Monitoring and Sampling Activities--The observations and tests that will be used to ensure that the construction or installation meets or exceeds all design criteria, plans, and specifications for each hazardous waste management unit component should be described in the CQA Plan. The sampling and monitoring activities for all constructed components, sample size and sample locations, frequency of testing, data evaluation procedures, acceptance and rejection criteria, and plans for implementing corrective measures as addressed in the project specifications should all be presented in the CQA Plan.

- Documentation of Construction Quality Assurance Activities--Reporting requirements for CQA activities should be described in detail in the CQA Plan at the time of its submittal for approval. This should include such items as daily summary reports, observation data sheets, problem identification and corrective measures reports, block evaluation reports, acceptance reports, and final documentation. Provisions for the final storage of all records should also be presented in the CQA Plan.

Each of these elements is described in the following subsections. A detailed outline of the requirements of the CQA Plan is presented in USEPA [1986b], from which these requirements have been adapted.

### 4.2.1 General Description of the Unit

The land disposal unit should be clearly identified in the CQA Plan, including direct references to the project plans and specifications, and a description of the type of waste management unit, its dimensions and capacity. The proposed operating duration, and plans for closure must also be presented.

### 4.2.2 Responsibility and Authority

#### 4.2.2.1 Organizations Involved in CQA

The principal organizations involved in permitting, designing, and constructing a hazardous waste management unit include the Permitting Agency or Regulator, the unit Owner/Operator, the Designer, the CQA Contractor, and the Construction Contractor(s). Except for the Regulator, the principal organizations will not necessarily be completely independent of each other; e.g., the unit Owner/Operator may also be the Construction Contractor, and the CQA Contractor personnel may be employees of the unit Owner/Operator, the Designer, or an Independent Firm. Regardless of the relationships among the organizations, it is essential that the areas of responsibility and lines of authority for each organization be clearly delineated early in the CQA Plan. This will help establish the necessary lines of communication that will facilitate an effective decision-making process during implementation of the site-specific CQA Plan. It is also essential that the organization performing CQA operates independently of and is not responsible to the organization(s) involved in constructing the unit.

#### 4.2.2.2 Project Meetings

Periodic meetings held during the life of the project strengthen responsibility and authority by enhancing communication between personnel responsible for designing, constructing, and documenting construction of a hazardous waste management unit. Since conducting periodic project meetings is not a mandatory feature of the proposed Liner/Leak Detection Rule, the decision to hold project meetings is at the option of the unit Owner/Operator; however he may delegate that responsibility to one of his supporting organizations (e.g., Designer or CQA Contractor). Regardless of which party conducts them, periodic project meetings benefit all those involved with the unit by ensuring familiarity with unit design, construction procedures, and any design changes. Examples of the types of meetings that may be held are:

- Preconstruction CQA Meeting

  Immediately prior to construction, a meeting may be held to resolve any uncertainties following the completion of the unit design, completion of the site-specific CQA Plan, and award of the construction contract. The unit Owner/Operator, Designer, CQA personnel, and Construction Contractor would normally all be present. This meeting would serve to familiarize all parties with the CQA Plan, their various responsibilities, lines of authority, etc., prior to the commencement of any site construction activity. The meeting would also resolve differences between the parties in sufficient time to allow modifications to the CQA Plan prior to commencement of construction.

- Daily Progress Meetings

  A progress meeting may also be held daily at the work area just prior to commencement, or following completion of work. This meeting would normally be attended by the Construction Contractor and the CQA personnel. The purpose of the meeting would be to review the activities of the previous day or shift, review the activities for the upcoming day or shift, and discuss any potential construction problems.

- Problem or Work Deficiency Meetings

  A special meeting may be held when and if a problem or deficiency is present or likely to occur. At a minimum, the meeting would be attended by the Construction Contractor and the CQA personnel. The purpose of the meeting would be to define and resolve a problem or recurring work deficiency.

### 4.2.3 Personnel Qualifications

The CQA Plan should identify the required qualifications of the CQA Officer and the CQA Team, and describe their expected duties.

#### 4.2.3.1 CQA Officer

The CQA Officer is assigned responsibility for all aspects of CQA Plan implementation. The CQA Officer is responsible to the unit Owner/Operator, and should function independent of the Construction Contractor. The location of the CQA Officer within the overall organizational structure of the project, including the unit Owner/Operator, Designer, Construction Contractor, and Regulator, should be clearly described within the CQA Plan as noted in the previous discussion on responsibility and authority.

The CQA Officer should possess adequate formal academic training in engineering, engineering geology, or closely associated disciplines; and sufficient practical, technical, and managerial experience to successfully oversee and implement Construction Quality Assurance activities for hazardous waste management units. In almost all states, the responsibilities of a CQA Officer are of a nature that lead to the requirement that he/she be a registered Professional Engineer. Because the CQA officer may have to interrelate with all levels of personnel involved in the project, good communication skills are essential. The CQA Officer should ensure that communication of all CQA-related matters is conveyed to and acted upon by the affected organizations. The CQA Officer should have specific training, experience, and knowledge of the materials (soils and geosynthetics) for which he will be providing Construction Quality Assurance.

On some projects (particularly large ones), the duties of the CQA Officer may be divided between two or more persons. For example, the engineering aspects may be performed by a Managing Engineer, while the construction and managerial aspects may be performed by an on-site Quality Assurance Manager.

#### 4.2.3.2 CQA Monitoring Personnel

The CQA Monitors should possess adequate formal training and sufficient practical, technical, and administrative experience to execute and record CQA activities successfully. This should include demonstrated knowledge of specific field practices relating to construction techniques used for hazardous waste management units, all codes and regulations concerning material and equipment installation, observation and testing procedures, equipment, documentation procedures, and site safety. They should also have specific training and knowledge regarding the materials (soils and geosynthetics) which they will be monitoring.

#### 4.2.3.3 Consultants

Authorities in engineering geology, geotechnical engineering, civil engineering, and other technical disciplines may be called in from external organizations in the event of unusual site conditions or observations. The final report should present detailed documentation of Consultant qualifications when expert technical judgments are obtained and used as a basis for decision in some aspect of Construction Quality Assurance. Expert opinions should not be used as a substitute for objective data collection and interpretation when suitable observations and test procedures are available.

### 4.2.4 CQA Monitoring and Sampling Activities

The CQA Plan should describe the monitoring and sampling activities (observations and tests) that will be performed by the CQA Team during hazardous waste management unit construction. The scope of this discussion should address only the construction and installation of all unit components and the manufacture/fabrication of various components and subcomponents when pertinent. It is assumed that the site has been characterized adequately, including evaluation of the hydrogeologic environment. It is also assumed that a site-specific unit design has been prepared that meets regulatory requirements and is acceptable to the unit Owner/Operator, and that this design has been evaluated to ensure its technical correctness and feasibility.

The components of the unit which require the provision of CQA services include (but are not limited to):

- foundations;
- dikes;
- compacted low-permeability soil liners;
- other soils;
- flexible membrane liners;
- leachate detection, collection, and removal systems;
- other geosynthetics; and
- final cover systems.

For many materials and construction processes, it is necessary to estimate the quality of the overall material or process from the observed or measured quality of a representative sample that is a small fraction of the total material or process. Examples of these situations include assessment of characteristics of a compacted soil liner (e.g., permeability, moisture content, density, particle size distribution) and destructive testing of FML seams.

Some of the key characteristics of commonly used sampling strategies include:

- data type;
- acceptance/rejection criteria;
- sampling units;
- number of sampling units and number of measurements per unit;
- location of sampling units and/or measurements within units;
- treatment of outliers; and
- corrective measures.

The current state of knowledge on sampling strategies for hazardous waste management unit CQA is not well enough developed to enable EPA to recommend a specific approach for designing a sampling strategy. For instance, the measurement error inherent in test methods is an important piece of information when devising a statistical sampling strategy. However, the measurement error associated with certain important test methods (e.g., laboratory and field permeability) is not known. Until more information is available, the selection of appropriate sampling strategies should be conducted with the guidance of knowledgeable engineers and statisticians.

### 4.2.5 Documentation of Construction Quality Assurance Activities

The ultimate value of a CQA Program depends to a large extent on recognition of all of the construction activities that should be monitored, the selective assignment of responsibilities to the various members of the CQA Team for the monitoring of each activity, and the careful documentation of all observations. This documenting of CQA activities must be addressed in the CQA Plan. The CQA Team should be reminded of the items to be monitored, and should note, through required descriptive remarks, data sheets, and checklists signed by them, that the monitoring activities have been accomplished.

#### 4.2.5.1 Daily Record Keeping

Standard daily reporting procedures should include the preparation of a summary report with supporting logs and data sheets and, when appropriate, problem identification and corrective measures reports. In particular, the following documentation should be prepared on a daily basis:

- logs;
- data sheets;
- problem identification/corrective measures reports;
- photographic reporting data sheets;
- documentation of correction (cross-referenced to data sheets);
- final results;
- suggested methods to prevent similar problems; and
- signature of the appropriate CQA monitoring personnel and concurrence by the CQA Officer.

In some cases, not all of the above information will be available or obtainable. However, when available, such efforts to document problems could help to avoid similar problems in the future.

Upon receiving the CQA Officer's written concurrence, copies of the report should be sent to the Designer and the unit Owner/Operator for their comments and acceptance. These reports should not be submitted to the Permitting Agency at that time unless they have been specifically requested. However, a summary of all data sheets and reports may be required by the Permitting Agency upon completion of construction.

### 4.2.5.2 Photographic Reporting Data Sheets

Photographic reporting data sheets may also prove useful. Such data sheets could be cross-referenced or appended to data sheets and/or problem identification and corrective measures reports. These photographs will serve as a pictorial record of work progress, problems, and corrective measures.

### 4.2.5.3 Block Evaluation Reports

Within each construction block (a block is a group of related activities that are carried out at a certain point in time), there may be several quality characteristics, or parameters, that are specified to be observed or tested, each by a different observation or test, with the observations and/or tests recorded on different data sheets. At the completion of each block, these data sheets should be organized into a block evaluation report. These block evaluation reports may then be used to summarize all of the site construction activities.

### 4.2.5.4 Acceptance of Completed Components

All daily summary reports, data sheets, problem identification and corrective measures reports, and block evaluation reports should be reviewed by the CQA Officer. The documentation should be evaluated and analyzed for internal consistency and for consistency with similar work. Timely review of these documents will permit errors, inconsistencies, and other problems to be detected and corrected as they occur, when corrective measures are easiest.

The above information may be assembled and summarized into periodic Acceptance Reports, or otherwise summarized in the final CQA

Report. The reports should indicate that the materials and construction procedures comply with the plans and specifications. These reports should be included in project records, submitted to the unit Owner/Operator, and submitted to the Permitting Agency.

#### 4.2.5.5 Final Documentation

At the completion of the project, the unit Owner/Operator must submit a final report to the Permitting Agency. This report should include all of the daily summary reports, data sheets, problem identification and corrective measures reports, block evaluation reports, photographic reporting data sheets, acceptance reports, deviations from design and material specifications (with justifying documentation), and record drawings. This document should be signed by the CQA Officer and included as part of the CQA Plan documentation.

#### 4.2.5.6 Document Control

The CQA Plan and all CQA documentation should be maintained under a document control procedure. This indexing procedure should provide for convenient replacement of pages in the CQA Plan, thereby not requiring a revision to the entire document; should identify the revision status of the CQA documents; and should enable the CQA documents to be organized in terms of their relationship to each other, the CQA Plan, and the time and location of the materials and/or workmanship that they represent.

#### 4.2.5.7 Storage of Records

During the construction of a hazardous waste management unit, the CQA Officer should be responsible for all unit CQA documents. This includes the CQA Officer's copy of the design criteria, plans, and specifications, the CQA Plan, and the originals of all data sheets and reports. It is suggested that duplicate records be kept at another location to avoid loss of this information if the originals are destroyed.

Once unit construction is complete, the document originals should be stored by the Owner/Operator in a manner that will allow for easy access while still protecting them from any damage. An additional copy should also be kept at the unit if this is in a different location from the Owner/Operator's files. A final copy will be kept by the Permitting Agency in a repository accessible by the public. All documentation should be maintained through the operating and post-closure monitoring periods of the unit.

## 4.3 NEED FOR A CQA PROGRAM

### 4.3.1 Background

The performance of a lining system is sensitive to both the methods and procedures followed during construction as well as to the quality of materials used to construct it. Once constructed, each component must be able to perform its intended function within the lining system. This need has been recognized by the EPA, and resulted in the preparation of a preliminary technical guidance [USEPA, 1987], and finally a Technical Guidance Document, "Construction Quality Assurance for Hazardous Waste Land Disposal Facilities" [USEPA, 1986b]. In this document, the background information provided identifies those parameters which are critical to the successful completion of the unit. The result is that CQA has been incorporated as an integral component of the proposed Liner/Leak Detection Rule. As component design criteria and construction methods improve, the CQA Program should provide a means of monitoring and documenting them. By doing so, the CQA Program will become a part of the cost of the facility. The following sections examine the considerations and constraints that cumulatively illustrate the need for the CQA Program.

### 4.3.2 Effect of Construction Procedures on Lining System Performance

The methods and procedures used to construct all components of landfills, surface impoundments, land treatment units, and waste piles directly affect the performance of the respective components of the facility. In addition, those methods and procedures used to construct a particular component of the system may, either directly or indirectly, affect the performance of other components of the facility.

Certain types of problems may potentially arise which must be specifically addressed, in order to ensure conformance to the plans, specifications, Construction Quality Assurance Plan, and good construction practice in general. The objective of the CQA program is to help minimize the occurrence of these potential problems. The problems, which can be divided into soils-related problems and geosynthetics-related problems, are reviewed below to demonstrate potential benefits arising from the CQA program.

### 4.3.2.1 Soils-Related Construction Problems

#### 4.3.2.1.1 Moisture Control and Compaction

Compaction criteria which are developed for the placement of soil materials, primarily for site earthworks, but also for the compacted soil components of the lining system, are generally based on one of two standard laboratory tests: ASTM D698 "Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures, Using 5.5-lb (2.49-Kg) Rammer and 12 in (305 mm) Drop"; and ASTM D1557 "Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10 lb (4.54 Kg) Rammer and 18 in (457 mm) Drop" (see also Section 4.5.1.1.1). Commonly referred to as the Standard Proctor and Modified Proctor tests, respectively, these tests produce, for a given soil, the relationship between achievable dry density and moisture content and a particular level of compactive effort. The moisture content for which the highest dry density (referred to as the maximum dry density) is achieved, is called the optimum moisture content. The field compaction specification is then based on a given percentage (usually 90 percent to 100 percent) of this maximum dry density and on a given range of water contents referenced to the optimum water content. Density is specified in cases wherein strength or soil support is a primary requirement for the compacted soil. In other situations, such as for the compacted soil components of the liner systems, for which low permeability is the requirement, specifications may call for a particular hydraulic conductivity, regardless of the compaction of the soil. In fact, optimum conditions for maximizing soil density are not necessarily the same as optimum conditions for minimizing soil permeability.

Compaction testing criteria, when applied to site earthworks such as berms or other site soils which are not components of the lining system, are an effective indicator of conformance to the specifications. For the compacted soil liners, however, it is hydraulic conductivity which is the specified property. The density and water content of the soil can, however, still be used as an approximate indicator of the permeability, provided that the relationships between the degree of compaction, density, moisture content, and hydraulic conductivity are known. These relationships are discussed further in other sections of this chapter.

Very few soils are indifferent to their moisture condition, in regard to the maximum dry density. In fact, the characteristic curves (Figure 4-1) taken from the results of either Proctor test show much lower values of attainable density (for that given, fixed level of compactive effort) at moisture contents both lower than, and higher than optimum (curve 1). As can be seen, however, for increasing levels of compactive effort (curves 2 to 4), the attainable density of the soil increases, while the optimum moisture content decreases. A line can be drawn through the 'family' of maxima, which is indicative of the universe of maximum densities for different levels of compactive effort. For saturated samples, this line would coincide with the zero air voids line, shown on the figure.

It is known, however, that the hydraulic conductivity of clayey soils is not directly related to dry density. There has been considerable research into this phenomenon, the reasoning is well documented, and a number of theories exist to explain this fact. [Bjerrum et al., 1957; Lambe, 1954; Lambe, 1958; Mitchell et al., 1965; and Seed et al., 1959; Hilf, 1975]. Hermann and Elsbury [1987] present a recent review of theories related to the structure and properties of compacted soils.

One theory commonly used to explain the relationship between water content, dry density and hydraulic conductivity is the flocculated versus disperse structure theory of Lambe [1958]. In short, clay particles are relatively flat, and in effect are essentially two-dimensional. At low moisture contents, and in a relatively loose state, they generally have a flocculated structure, with nearly random particle orientations within the flocs. At higher moisture contents, clay particles tend to orient themselves preferentially in one direction along their two-dimensional plane. This is called dispersion. It is known that for a given compactive effort, the tendency of a clay to disperse during compaction is greater at higher moisture contents, and that, due to the ability of the particles to fit much more closely together, lower hydraulic conductivities are possible at higher moisture contents. At the top of Figure 4.2 is a curve representing the degree of dispersion of the clay. This curve tends to show that for a given degree of compaction, at higher levels of dispersion, and higher moisture contents, the attainable hydraulic conductivity is much lower than at lower dispersion levels, and at lower moisture contents.

Figure 4.2 is an example which, within reasonable limitations, illustrates the 'typical' case for a clay soil. As can be seen, the moisture content for a minimum hydraulic conductivity is higher than the optimum compaction moisture content. Additionally, considerable variation in hydraulic conductivity results from different moisture conditions of the soil, given the same level of compactive effort. The variation in hydraulic conductivity spans two or more orders of magnitude, whereas the difference in dry density for different moisture contents is relatively small. In fact, the ratio of maximum to minimum hydraulic conductivity is about 100, whereas the ratio of maximum density to minimum density is about 1.1, over the same range of moisture contents, and with the same level of compactive effort. [Mitchell et al., 1965].

From the above, it is apparent that the required moisture content to meet a compaction criterion is generally dry of the moisture content required to meet a permeability criterion. Because of the relationship illustrated in Figure 4-2, it may be possible, however, to use the compacted dry density and moisture content in the fill as an indicator of hydraulic conductivity. This correlation will normally be determined during the Test Fill program. This is discussed further in Sections 4.3.2.1.11 and 4.5.1.4.

For earthworks for berms and other structural soil components of the unit, the dry density and moisture content compaction criteria are most appropriate for specification and conformance testing. For example, Figure 4-3 shows an example moisture-density relationship for the compaction of a soil. If, for this soil, a specification of 95 percent of the maximum dry density is desired, then it can be seen that this should be attainable if the moisture content of the soil falls within the range $w_{min(95\%)}$ to $w_{max(95\%)}$, for a degree of compaction that is consistent with the laboratory test. Therefore, for the modelled level of compactive effort, an area of the fill with a moisture content above $w_{max(95\%)}$ or below $w_{min(95\%)}$ will require moisture conditioning (wetting or drying, as required).

The composition of the soils in the field dictate the required ranges in moisture contents and compactive effort to achieve the specified dry density. Although soils vary in constituents, mineralogy, and moisture condition from one location to another, several general guidelines can be applied to field compaction of soils:

- clays and other fine grained soils, which would typically be used in the soil component of a liner, often have natural moisture contents, in cold or very wet climates, between 5 and 20 percent in excess of their optimum moisture content; in hot or drier climates, natural moisture contents on the dry side of optimum are more usual;

- the range of moisture contents over which a given degree of compaction can be attained is generally smaller for fine grained soils (silt, clay) than for granular soils (sand, gravel); and

- granular soils above the ground-water table commonly have natural moisture contents dry of the optimum moisture content, depending on local drainage, fines content, and other considerations.

In any event, when a high degree of compaction (say, greater than 95 percent) is required, moisture conditioning, in the form of wetting or drying, is often required for both fine grained and coarse grained soils. This poses a considerable constraint on the scheduling and cost of earthworks, including installation of the compacted soil component of the lining system. The quality control testing to confirm that the specifications have been attained is therefore an important component of the Construction Quality Assurance Program. Various well-established test procedures exist (See Section 4.5.1), which are reliable and easy to perform. The measurements of dry density and moisture content do not, in themselves, represent a problem. Considerable difficulty does exist, however, with the moisture conditioning approach, and close Construction Quality Assurance is necessary to confirm the uniformity and preservation of the drying or wetting process.

#### 4.3.2.1.2 Weather and Climate

Closely related to the quality assurance of compaction are the climatic constraints imposed by virtue of the geographic location of the site. High-quality earthworks (i.e., soils placement and compaction to stringent specifications) cannot reliably be attained in below-freezing (<0°C or 32°F) temperatures. Seasonal constraints will therefore limit field operations in major geographic portions of the United States.

Precipitation also impacts soils placement and compaction. The tolerable range of moisture contents noted in 4.3.2.1.1 is often difficult to meet in very wet or very dry climates. Excessive wetting through precipitation and subsequent ponding can result in softening of both in-place and stockpiled materials. This may necessitate removal of portions of the soils that have already been placed and compacted. Because hydraulic conductivity is more affected by changes in compaction moisture content than is dry density, the moisture requirements for soil liners are more stringent than for general earthworks, and hence this problem is considerably more prevalent for compacted soil components of liners. The properties of clay soils which make them particularly effective for low-permeability liners also make them sensitive to compaction moisture content. Placement of these clays at moisture contents wet of optimum is typically required to allow attainment of the specified hydraulic conductivity criterion. The Test Fill program will be able to identify the relationship between the compaction criteria and the specified hydraulic conductivity (see Section 4.4.2.2).

Similarly, excessive drying of the soils due to the hot summer sun, high winds, and/or climates with low relative humidity can cause problems not only during, but subsequent to placement. Fine grained soils such as clays are frequently very difficult to moisten in a uniform manner. Soils which will meet the low-permeability criterion for compacted soil liners typically exhibit high plasticity, and hence a propensity for volume change in response to a change in moisture content. Consequently, drying-out of these soils after placement and compaction may result in the formation of desiccation cracks, which can be as much as 25 to 50 mm (1 to 2 in.) wide and up to 300 mm (12 in.) deep. If this condition is encountered, the soil will require scarification and recompaction over the entire depth of desiccated soil.

The problems of hot, cold, wet, or dry periods will, to a greater or lesser degree, prove to be a construction constraint on almost every project. The Construction Quality Assurance of the earthworks is therefore essential, because contractors are often anxious to compensate for weather-related delays and may be inclined to relax their standards in the absence of a stringent Construction Quality Assurance Program.

#### 4.3.2.1.3 Availability of Suitable Soils

Earthworks soils can frequently be specified to accommodate those materials excavated from the site. In this manner, soil materials handling can be optimized, and costs minimized. The requirements for compacted soil components of lining systems, however, are much more inflexible. As discussed in other chapters, the principal requirement of the compacted soil liner is specified as a maximum permeability criterion ($10^{-9}$ m/s ($10^{-7}$ cm/sec)). Soils which will satisfy this requirement may be unavailable in many locations. Similarly, granular drainage media within the lining system have a minimum planar permeability requirement of $10^{-2}$ m/s (1 cm/sec). This represents a clean, coarse-grained sand, and availability can also be a problem. The Construction Quality Assurance Program is a useful mechanism to ensure that proposed imported fine-grained soils and granular drainage materials have properties which will enable them to meet the specifications.

#### 4.3.2.1.4 Subgrade Soils

The CQA Program can ensure that site selection process precludes the location of landfills, surface impoundments, waste piles, and land treatment units on sites with organic soils, badly fractured rock or other undesirable foundation materials. Lining systems in which the secondary liner rests directly on the recompacted subgrade soil will be construction-sensitive to the condition of the subgrade immediately prior to placement of the FML. Regardless of the soil types existing at the site, the compaction of these soils must be properly completed in order to attain the required support and surface. In some cases, soils may even have to be imported from off-site, in order to obtain specified compaction, and a proper surface and liner support.

#### 4.3.2.1.5 Soils Homogeneity and Layering

In nature, the sedimentary process deposits soils gradually and preferentially (coarse-grained materials first, fine-grained last). Therefore, any given source of clay may exhibit varying properties. No matter what level of care is taken in placing soils in lifts, there may be zones of non-homogeneity within a soil lift and at the interfaces of adjacent lifts. Lift thicknesses are selected based on the soils conditions and the nature of the compaction equipment being used. The desire for thick lifts must be weighed against the ability to compact the material to a relatively uniform density and permeability throughout the lift. Consequently, soil lift thicknesses will tend to be in the 150 to 250 mm (6 to 10 in.) range. From compaction and permeability viewpoints, this provides a "relatively" homogeneous soil mass, in which the degrees of compaction and permeability are acceptably uniform throughout, and the sizes of soil clods minimized. It should be recognized that, from a permeability viewpoint, a lift-to-lift interface or interlayer zone is created that will differ in some manner from the other portions of the fill. For example, the homogeneity of the soil will be reasonably consistent, but the horizontal and vertical hydraulic conductivities of the soil within this zone may vary.

The potential problem of a preferential flow path, created by a potentially higher lateral permeability along the layer interface, can be minimized by the scarification of the top 50 to 75 mm (2 to 3 in.) of the previous lift, immediately before the placement of a new lift. This allows better intermixing of the two layers and breaks up any desiccated crust (which can form after only a few hours exposure to sun and wind) on the previous lift.

#### 4.3.2.1.6 Disturbance Due to Traffic

Disturbance due to traffic is wholly preventable, but still sometimes occurs. The concentrated loads imposed by vehicles can severely rut the surface of compacted soils. Remedial measures required will vary depending on the soils, nature of the disturbed area, and severity of the damage, and can range from minor dressing, to excavation and removal. Control of all construction traffic, and the prevention of access to finished lifts or areas, will preclude this particular problem from arising.

#### 4.3.2.1.7 Soil Components of Top Composite Liners

It should be noted that soil components of top composite liners will be typically placed over one or more geosynthetic layers, and care must be taken to ensure that no damage occurs to the geosynthetics during the placement and compaction of these overlying soil layers. As a consequence, it is usually preferable to not specify compaction or permeability requirements for the bottom-most lift of the soil component of the top composite liner. It is better to simply place this lowest layer with low ground pressure tracked equipment and no compaction equipment. Additionally, the specification should require the first lift over underlying geosynthetics to be a minimum of 300 mm (12 in.) thick. The specification for soil components of top composite liners should therefore require the first lift to be a minimum of 300 mm (12 in.) thick of relatively uncompacted soil. (This lift could even be granular material if available clays are too difficult to place without compaction equipment.) Overlying lifts can receive increasing degrees of compaction and the top lift should be fully compacted and smooth rolled to act as the surface upon which the FML component of the top composite liner will be placed.

#### 4.3.2.1.8 Testing Problems - Field Compaction of Soils

The current state of practice regarding the field compaction testing of soils is relatively sophisticated. Consequently, few problems are encountered with the testing of soils for density determination. Nuclear density gauges are typically used, and these devices provide both density and moisture content to depths up to 200 mm (8 in.). These apparatus are simple to use, accurate, and fast. As a result, the productivity of CQA personnel using this equipment can be high, producing many tests in a shift. But more importantly, nuclear density gauges provide CQA personnel with an immediate answer. Other more traditional procedures, such as the sand cone test, although reliable, take longer to perform, and require overnight laboratory determination of moisture content, and are sensitive to the care taken by, and the experience of, the CQA technician. It is appropriate, however, to cross reference and correlate the nuclear test results with periodic sand cone testing.

#### 4.3.2.1.9 Testing Problems - Laboratory Moisture-Density Tests

Laboratory moisture-density relationships for all soils used at the site are carried out in conformance with either the Standard or Modified Proctor tests (see Section 4.5.2.1.1). The moisture-density tests provide the 'maximum' (for the given level of compactive effort) dry density attainable, and the corresponding moisture content. These tests are relatively easy to perform, and provide the basis for the evaluation of the relative degree of compaction of the soil, i.e., they are the basis for the in situ tests discussed in the previous section.

Few problems exist with these tests, although frequently some very highly plastic clays will have optimum moisture contents very much higher than the natural, in-place moisture content, and are difficult to work with in this wet condition. In that instance, the performance of the test may be very difficult, and may be a function of the technician's experience and ability, and laboratory technique.

In addition, some uniform sandy soils are very moisture content insensitive, and virtually the same density may be obtained over a range of moisture contents. In this case, the test is still relatively easy to perform, however the interpretation of the results is sometimes difficult. In general, however, an experienced technician performing the test can alleviate or overcome this problem.

#### 4.3.2.1.10 Testing Problems - Laboratory Permeability Determination

At present, laboratory permeability testing for Construction Quality Assurance purposes is carried out on samples of the compacted soil liner and sand drainage layers in the absence of generally accepted in-place procedures which can be performed relatively quickly. Several procedures exist for the performance of permeability testing in the laboratory (see also Section 4.5.1.1.3). Constant head tests (in which flow is maintained through the soil under a fixed level of head) and falling head tests (in which an initial head of water is not replenished as the water flows through the sample) can be performed in each of the following apparatus:

- Soil Permeameter;
- Oedometer Cell;
- Triaxial Cell; and
- Rowe Cell.

Each of these apparatus are fairly well developed as a laboratory tool for the determination of soil hydraulic conductivity. The nature of the actual test performed (constant head or falling head) is a function of the nature of the soil.

A recurrent problem related to laboratory testing of this nature is the sensitivity of the test result to sample disturbance. Cohesive soils (e.g., clay) can be collected in a 'relatively' undisturbed state, however problems related to handling, transport, extrusion, and sample preparation introduce an unquantifiable error to the test. Fairly stringent permeability requirements are applied to clay components of lining systems ($10^{-9}$ m/s ($10^{-7}$ cm/sec)) and granular components of drainage systems ($10^{-2}$ m/s (1 cm/sec)). As a result, the test can be properly carried out and a result obtained, but the representativeness of the test result to the field conditions (the in situ permeability) is uncertain. In fact, sample disturbance can cause tests to err in either direction from the "correct" result.

Cohesionless soils are especially difficult to represent in the laboratory and, in fact, laboratory permeability tests on granular soils will almost always have to be performed on reconstructed samples. In this case, clearly, the suitability of the test result as an indication of field permeability is in question.

In spite of these problems, this mode of testing is still acceptable in that the quality control of the test itself is very reliable. In each case, a test result can be obtained which can be considered to be representative, within an often unspecified margin, of the permeability of the sample in its as-tested condition. The interpretation of the relationship between the field condition and the sampled and tested condition remains the problem. At present, however, pending resolution of the issues in dispute over field permeability tests, laboratory testing will continue to be accepted, with some interpretation required.

### 4.3.2.1.11 Testing Problems - Field Permeability Determination

Field testing of soil permeability is currently in the developmental stage. Several procedures and apparatus have been developed [Daniel, 1984; Boynton and Daniel, 1985; Daniel et al, 1985], but are not yet in common usage and, certainly, no standards yet exist. The procedures which have been proposed for the determination of field permeability (hydraulic conductivity), are thoroughly documented, but sufficient studies of the accuracy and relevance of the test results have not yet been carried out.

The results of field hydraulic conductivity tests often yield higher values than the laboratory tests (by up to as much as two orders of magnitude, or 100 times) [Daniel, 1984; Daniel et al, 1985]. Considerable discussion over the meaning of this variation has been carried out. In the case of careful prevention of sample disturbance (by the collection of block samples, for example) the resultant laboratory hydraulic conductivity will be representative of the field condition on a micro-scale. In contrast, field tests carried out on larger 'sample areas' incorporate the effects of desiccation, secondary structure, rootholes, etc. In the case of a prepared fill for a compacted soil liner for which CQA has been provided, many of these anomalies will be minimized, and the assumption of a homogeneous soil mass, as discussed previously, may be more closely met. In general, however, the measured laboratory hydraulic conductivity will tend to be lower than the field measurements of hydraulic conductivity.

The problem with field hydraulic conductivity testing at present, and the reason that it cannot be used for quality control purposes in its present form, is that it can take up to 3 to 4 months to obtain results which are considered to be representative (due to the time factors associated with saturation and soil swelling). Among the methods now available, some apparatus which provide results in shorter times are being evaluated. The nature of this equipment and these test methods is discussed in Section 4.5.1.1.4 of this document.

As a means of reconciling the time delays associated with the field tests and the potential inaccuracy of the laboratory tests, the current state of practice calls for the use of field tests on Test Fills in order to determine the suitability of available materials and placement techniques, and corrolate these data to moisture/density relationships, followed by laboratory testing of the actual liners (see Section 4.5.1.1.4).

#### 4.3.2.2 Geosynthetic-Related Construction Problems

##### 4.3.2.2.1 Manufacturing Quality Control

Manufacturers of geosynthetics (including flexible membrane liners, geotextiles, geonets, geogrids, etc.) provide in-house quality control of their manufacturing process, in part due to the need to confirm that the process is functioning properly, and in part to provide documentation and certification that the properties specified for the material for the specific project have been satisfied.

Nevertheless, in some cases, substandard materials, which do not meet the project specifications and must therefore be rejected, have been shipped to a project site. If a thorough CQA program is instituted on site, the performance of conformance tests (see Section 4.5.2.1) will intercept these materials, but the cost of rejection can be high if construction delays result from re-ordering and remanufacturing of materials. For this reason, CQA of the manufacturing process can play an important delay and cost-saving role in the project, in addition to confirming the adequacy of the materials.

##### 4.3.2.2.2 Fabrication Quality Control

Geosynthetic products are sometimes prefabricated prior to shipment to the site, and this may either take place at the manufacturing facility or at some other location. This fabrication process typically consists (in the specific case of FML's) of the seaming together of rolls and portions of rolls into larger panels, to facilitate placement at the site. Although the conditions under which this fabrication seaming takes place are normally much better than in the field (e.g., indoors, flat surface, etc.), this is still a most critical operation. Fabrication seams are not always tested once the panels are deployed in the field; therefore the CQA Program should include the material fabrication process in order to thoroughly review all of the methods and procedures, and to monitor and document the testing on these materials.

### 4.3.2.2.3 Shipping and Handling

Geosynthetics, particularly geotextiles and flexible membrane liners, are very susceptible to damage if improperly shipped, stored, or handled. In addition, their performance is, at least in part, dependent on the continuity of the sheet material, and damage occasioned by mishandling or other poor practices which will require repair. Geonets are somewhat more forgiving by the nature of their function, and damage such as a hole or a tear can more easily be repaired, or the damaged section removed, without disruption of the integrity of the remainder of the roll. At the other extreme, geogrids which are required for reinforcement must be continuous in the direction of the applied load. Consequently, damage to rolls of geogrid will normally result in rejection of the entire roll (unless an alternate location requiring a shorter length is available).

Geotextiles and FML's must be protected from dust, dirt, moisture, and ultraviolet radiation during shipping, storage, and handling. Protective wrappings should be provided, in order to alleviate such problems. Geonets must be protected from dust and dirt and protective wrappings should therefore be provided for them also.

Handling problems usually relate to damage caused by handling equipment, such as forklifts or loaders, which can puncture or tear the rolls while picking them up and transporting them. In addition, a common source of geosynthetics damage is abrasion caused by dragging rolls or panels of material when positioning them for deployment. This can cause severe damage, and frequently can wear completely through one or more layers on the roll. Damage of this nature may not be repairable, in which case the damaged material should be rejected.

Storage of the geosynthetics on site, prior to deployment and installation, requires a location in which damage or contamination due to wind, dust, dirt, rain, and ultraviolet exposure is not possible. Particularly in the case of geotextiles which are to perform filter and drainage functions, contamination and clogging by dust or mud may well be irreparable. Most other geosynthetics (e.g., FML's, geonets, and geogrids) can usually be cleaned by washing.

The provision of Construction Quality Assurance for these components of construction can effectively minimize the incidence of these problems.

#### 4.3.2.2.4 Sheet Material Defects

In general, sheet material defects will normally be intercepted at the manufacturing stage, if good quality control is present, and especially if Construction Quality Assurance is provided for that operation.

There are several types of flaws or defects which are found in homogeneous sheeting material for flexible membrane liners, and which are all a function of some irregularity or eccentricity of the manufacturing process. These include:

- pinholes, which are very small holes and are relatively rare, likely caused by moisture or inclusions in the system;

- holes, which can be up to 25 mm (1 in.) in size which could be caused by moisture, inclusions, or other manufacturing irregularity;

- blisters, which can be caused by the presence of vapors during the manufacturing process;

- craters, which are "holes" which have not completely penetrated the sheet, and which can be created by foreign matter;

- small bumps, which could be caused by excess concentrations of carbon black;

- insufficient thickness, caused by the feed process; and

- scratches or gouges, caused by impact or contact with external objects, which can result in weak points in the sheet.

Similar defects can occur in geotextiles. In addition, instances have been recorded in which needle-punched geotextiles have damaged the FML, or been damaged themselves, by the presence of broken needles within the fabric. These needles may be detected visually, and could possibly be found by the use of a metal detector used to sweep the geotextile surface.

In general, because of the structural nature of the materials, flaws in geogrid or geonet "sheet" materials would normally be relatively easy to detect visually.

It should be noted that flaws inherent in the material which do not manifest themselves by a visual indicator will be difficult to detect in the field. In fact, flaws such as insufficient thickness can only be detected at the edges of a roll or panel, using a micrometer or similar form of caliper. Thinness in the middle of a sheet cannot be measurably detected, except by sampling. Interception of these problems during the manufacturing process by observation, and close control of the extrusion process, would be the superior means of detection. In any event, the best assurance of detection of these types of flaws is to provide CQA at both the manufacturing and deployment stages of construction of the lining system. Even so, nondestructive test procedures are not presently available to allow really effective testing of the entire sheet surfaces. Of course, the CQA personnel can carry out sampling and destructive testing of the sheet material in areas which appear to be of concern, but good experience and judgment are required of the CQA personnel in order to recognize potential problem areas and then to differentiate between rejection, versus repair, of the panel or roll (excessive repairs are impractical and unwise).

### 4.3.2.2.5 Seaming Procedures

Frequently, the problems with seaming the geosynthetic materials dictate the amount of destructive testing that is carried out, and also highlight the value of CQA. Problems related to the seaming operations include:

- Flexible Membrane Liner Seaming

  The field seaming of the FML components of lining systems is perhaps the single most important activity for which the CQA Team provides monitoring and documentation services. There are many methods used to seam FML's, but, for purposes of this document, those methods and procedures utilized for the field seaming of polyethylene (PE) flexible membrane liners will be emphasized.

  There are two processes for seaming PE flexible membrane liners currently in common use. These procedures are:

  - fusion welding, with either a single track, or double track, in which the bond of the two FML sheets is achieved by the heating of the two sheets, and upon melting, the application of pressure to produce a homogeneous joint upon cooling and solidification; and

  - extrusion welding, consisting of an extruded hot polyethylene bead, which is either sandwiched between the sheets, or placed as a fillet weld at the edge of the upper sheet (extrusion welding is also used for repairs and spot patching).

  The seaming system used is normally either a proprietary one, or a Manufacturer/Installer-specific system. Regardless of the system, the seam that is prepared must exhibit, at the least, the following characteristics:

- the bond, or continuity of the seam, must be continuous over the entire length of the seam, and any discontinuity must be considered to be a flaw;

- the seam strength, or integrity, must be consistent and be within a tolerable statistical variation from the strength of the sheet itself; and

- the seam must be able to be formed in the field, under less-than-ideal conditions.

Under adverse weather conditions it is difficult to obtain a field seam which satisfies all three of the above-noted criteria. In order to determine the seam continuity and integrity, they are subjected to continuous nondestructive testing and selective destructive testing (see Sections 4.5.2.1.2 and 4.5.2.1.3). In general, the difficulties experienced with the formation of these seams, which result in inadequate seams include:

- in general:

  - moisture on the liner sheets can result in bubbles within the seam, which might result in low strength values and/or a bond break;

  - dirt or other foreign objects on the liner sheets can result in their inclusion within the fused seam; a discontinuity may be formed that results in low seam strength; and

  - very high sheet temperatures will result in many folds in the sheet due to thermal expansion, and cause the formation of 'fishmouths' along the edge, resulting in the need to cut and patch before seaming, as well as special seaming techniques.

- for fusion welding, in particular:
  - improper heat settings can result in lack of fusion due to the material not melting (setting not hot enough for the conditions), or can result in low strength values, or burning through, or inconsistent fusion (setting too hot for the conditions);
  - improper sheet overlap can result in trimming and consequent waste of material (if too much overlap, such that the hot wedge cannot hook over the lower FML), or loss of the double weld (if too little overlap, such that the track of the double weld is wider than the overlap); and
  - improper pressure settings can result in the squeezing out of the softened material (usually in hot weather, if too much pressure is applied), mechanical damage to the sheeting material (usually in cold weather, if too much pressure is applied), or lack of fusion (if not enough pressure is applied).
- for extrusion welding, in particular:
  - improper heat settings can result in a lack of fusion (setting not hot enough), or can result in uneven bond or burning through (setting too hot);
  - working in moist (e.g., morning fog or dew) conditions can result in moisture contaminating the extrudate supply, particularly if particulate resin is utilized; and
  - very low sheet temperatures can inhibit the proper fusion by cooling the extruded material too quickly.

Naturally, the combination of all of the site-specific conditions will tend to govern the incidence and degree of these potential problems.

The provision of high quality CQA for the seaming process is absolutely necessary. Many of these potential problems may not be detected by nondestructive test procedures, and it is necessary for the CQA Monitors to document the presence of contaminants or foreign objects, lack of sufficient overlap, moisture, or inadequate settings (via trial seams).

- Geotextile Seaming

Geotextile materials, whether used as transmission media, filters, separators, or cushions, should be seamed in the field. This seaming is achieved by sewing, fusion, or gluing. Various shapes of overlap and types of stitch or solvent are utilized, but there is usually no mechanical property criteria which must be satisfied to accept or reject the seam. Many Designers believe that sewing is the only acceptable means of seaming geotextiles. In that case, the CQA Monitor need only confirm the existence of the seam (in addition to the normal monitoring requirements - no dust, mud, damage, etc.). In the case of geotextile filters, fusion seaming is considered inappropriate because of the propensity for burning through the geotextile. Solvent seaming is usually not acceptable in hazardous waste management units.

- Other Geosynthetic Seaming

Geonets usually do not require seaming as there is usually no design load imposed within the plane of the material. Tying of geonets together to hold them in place and ensure continuity is usually the sole extent to which joining is attempted.

Geogrids require integrity in the direction of applied load (e.g., downslope). For this reason, geogrids should be placed in a single piece in this load direction, and it is preferable that they not be joined, although methods are available.

CQA efforts will, in these cases, be devoted to conformance testing, placement methods, and the documentation of continuity in the required orientation.

### 4.3.2.2.6 Seaming Constraints

Aside from those factors identified in Section 4.3.2.2.5 with respect to seaming procedures, other constraints apply with regard to the seaming of flexible membrane liners in particular. Weather, as with soils installation, poses the single greatest constraint on the construction operations.

Hot weather (i.e., ambient temperature in excess of 40°C (104°F) measured 150 mm (6 in.) above the liner) can be detrimental to flexible membrane liner installation operations, in that seaming equipment can damage the sheet materials. In addition, due to the black color of the FML, surface temperatures under hot weather conditions can reach 80°C (176°F), which makes work by installation personnel virtually impossible.

Cold weather (i.e., ambient temperature below 5°C (40°F)) makes seaming very difficult due to the seaming problems cited in Section 4.3.2.2.5; and special techniques, as well as considerable operator skill, are necessary in order to achieve adequate seams. In this particular case, with cold weather seaming, very thorough CQA is required for the seaming operation.

Seaming cannot be undertaken during any type of precipitation unless special measures are taken to protect the seams from moisture, humidity, and accelerated cooling.

#### 4.3.2.2.7 Contamination

One of the greatest problems with geosynthetics installation is the occurrence of prolonged periods of rainfall while the system is only partially completed. This invariably leads to some form of contamination of the lining system with respect to the intrusion of soil into the LDCRS at unsealed edges (not to mention damage to the underlying clay). In some cases, geotextile filters can be irreparably damaged. In any event, removal of some portions of the system, in order to obtain access to the contaminated materials is necessary. Washing or replacement of geonets and geotextiles is required, so that these components can operate in the manner intended. In extreme cases, clogging of drainage pipes and sumps could occur.

This is certainly one of the very important roles of the CQA Program, in that the degree of contamination can be examined and evaluated, and the repair documented for reference.

### 4.3.2.3 Qualifications of Personnel

The Construction Quality Assurance personnel must be capable of recognizing, interpreting, or otherwise identifying all of the potential problems which have been discussed herein. In order to fulfill this requirement, it is imperative that the CQA personnel be familiar with every aspect of the installation of geosynthetics and/or soils. In addition, these personnel must be familiar with the properties of each of the materials, as well as the tests which are required to confirm these materials properties. Finally, the CQA Team must have a strong backup engineering capability available in order to properly fulfill this role. This engineering capability, whether on or off site, must be thoroughly familiar with all aspects of the design of geosynthetic lining systems, and the properties and behavior of polymeric and soil materials.

#### 4.3.2.3.1 CQA Officer

The Construction Quality Assurance Officer should hold a B.S., M.S., or Ph.D. engineering degree and be registered as a Professional

Engineer (P.E.) in accordance with the laws of the state in question. He should be experienced with geosynthetics, including flexible membrane liners of the type specified on the project, geonets, geogrids, and geotextiles. He should also be experienced in quality assurance, particularly for the installation of geosynthetics and soils materials. The CQA Officer should be experienced in the preparation of quality assurance documentation, including quality assurance forms, reports, certifications, and manuals.

#### 4.3.2.3.2 CQA Manager

If the CQA Officer is not resident full-time on the site, he should designate a CQA Manager, who would be on site continuously, and should be specifically experienced in the installation of geosynthetics, including flexible membrane liners of the type specified for the project, geonets, geogrids, and geotextiles. In addition, he should be experienced in the placement and compaction of soils, particularly compacted soil components of lining systems. He should be specifically trained and certified by the CQA Contractor in the duties of a CQA Manager, and should be experienced in the quality assurance documentation required for installation of geosynthetics and soils. This CQA Manager performs those duties of the CQA Officer which involve on-site responsibilities.

#### 4.3.2.3.3 CQA Monitors

The CQA Monitors should be fully trained and/or experienced in the proper documentation of all CQA activities. In addition, CQA Monitors involved with the monitoring of soils placement and compaction should be fully trained and experienced in that aspect, and should be qualified and/or licensed to carry out the nondestructive testing required for the quality control of the placement of those materials. Similarly, CQA Monitors involved with the monitoring of the placement, seaming, and testing of the geosynthetics should be trained in those aspects of CQA.

### 4.3.2.4 Documentation of Problems

The literature documenting the problems with geosynthetics, in the form of case histories of failures, is somewhat sparse, because there has been a disinclination for the industry to document its problems. As a consequence, there have been reservations by some engineers about the effectiveness of these polymeric materials (also caused by human inertia with regard to new methods). Naturally, there is also a legal question with regard to the documentation of litigious issues. Several papers have been produced, however, which address the types of problems which have arisen with geosynthetics, and discuss the nature of the problems, with measures that can be taken to preclude those problems through design modifications or different construction practices [Bass et al., 1985; Ghassemi et al., 1986; Giroud and Goldstein, 1982; Giroud, 1984f; Giroud and Fluet, 1986; Morrison et al., 1981; and US EPA, 1986b]. The documentation of problems is discussed further in the context of the benefits of CQA in Section 4.3.4.3 of this document.

Soils problems are generally well documented, and in fact are covered in the geotechnical engineering programs at most universities. There are, however, a few aspects of the design of these land disposal units that distinguishes them from other engineering structures. In particular, the emphasis on permeability, as opposed to strength and density, as an installation criterion is not common. The solution of problems that are encountered is therefore dependent on the qualifications of the Construction Quality Assurance Team. These problems are documented in many sources, including [Turnbull et al., 1956; Hilf, 1957; Seed et al., 1957; Lambe, 1958a; Lambe, 1958b; Seed et al., 1959; Lowe et al., 1960; and Gibbs et al., 1960]. Considerable work has been carried out since the era of these references, however, they remain the cornerstone of research into the behavior of compacted clays. The reader is referred to US EPA [1986b], US EPA [1986c], and US EPA [1987] for contemporary discussions of the construction of, and CQA requirements for, compacted soil liners.

### 4.3.3 Materials Considerations

#### 4.3.3.1 Soils Materials Considerations

The soils components of land disposal units differ from the geosynthetic and other manufactured components in that, for the most part, they originate on site or near site. In fact, wherever possible, all of the soils used at the site should be from the site, from the viewpoint of handling, transport, and cost. Sometimes, however, some or all of the soils components may originate off-site.

From the viewpoint of the Construction Quality Assurance Program, it will be necessary for the Contractor to work with the soils, making the necessary changes (e.g., wetting or drying) in order to conform to the specifications. This places considerable demands on the CQA Team.

Even though soils placement and compaction are better understood than geosynthetics placement and seaming, the operations involving Construction Quality Assurance of the soils components of a unit can be more drastically affected by external factors, such as the weather, than for the geosynthetics.

#### 4.3.3.2 Geosynthetics Considerations

##### 4.3.3.2.1 Manufacturing Considerations

There are obvious issues affecting the Construction Quality Assurance operations, which relate to the manufacture of the geosynthetics (including flexible membrane liners, geotextiles, geonets, and geogrids). These issues are discussed as follows:

- Feedstock, Process, and Additives

  The resins used to manufacture flexible membrane liners originate from a relatively small number of suppliers. As a result, there is good control of the quality of the feedstock for the FML's. There are, however, considerable differences between the different resins. These differences will, of necessity, tend to make specification of base resins a significant issue.

- In-Plant Quality Control

The processes by which the flexible membrane liners are formed can be very closely controlled, and it can normally be expected that the quality of the sheet would be very uniform. Nevertheless, substandard materials are delivered to sites on a regular, although not frequent basis, which indicates that the system is not without its shortcomings. Although it is often not possible to conduct CQA during the manufacture of the site-specific materials, Construction Quality Assurance can be carried out during the manufacture of similar materials (and other geosynthetics, to a lesser degree of importance), thereby comprising a spot check of the process in general, as well as in-plant quality control procedures.

### 4.3.3.2.2 Fabrication Considerations

The issues which are of particular concern with regard to the fabrication of the rolls of geosynthetics into panels are generally the same as those for the manufacturing process. The two of primary concern are the quality control of the in-plant seaming operations, and the conditions under which the work is carried out.

- In-Plant Quality Control

The fabrication of the rolls of materials into panels is primarily applicable to the synthetic components of the lining systems. The apparatus and procedures used in the fabrication process are typically the same as are used in the field for field seams. The monitoring of the operations associated with this seaming should be as thorough as the field requirements, and ensure that all personnel and apparatus are in conformance with the Construction Quality Assurance Plan and the project specifications for those activities.

- Operating Environment

As an extension of the above point, even though the operating conditions for fabrication may be much more controlled than at the site (e.g., indoors), the degree of quality assurance required for the fabrication should justifiably be similar in scope. The external constraints such as the weather are more controlled for a fabricating plant, but the variables representing the problems which can arise are not different.

#### 4.3.3.2.3 Transportation and Handling Considerations

The transportation and handling of the geosynthetics, particularly the FML's and the geotextiles, are important to the preservation of the quality of the materials, since they are extremely fragile, and susceptible to damage, even in cases where proper care is being exercised. In general, specifications are written in such a manner that it is up to the Manufacturer to see that care is taken during handling, but often the Transporter is not conscious of the importance of protection of the materials. The specifier and the CQA Contractor should ensure that the handling criteria for the materials are specified and that these requirements are met.

### 4.3.4 Benefits of Construction Quality Assurance

### 4.3.4.1 Nature and Description of the Benefits

The benefits of a detailed CQA Program are often intangible, and in fact, as with any QA-related service, if the CQA Program is properly carried out, the benefits that can be directly attributed to that program may not be apparent at all. There are, however, some obvious benefits of the program that are real, regardless of the specifics of the work. These are discussed in the following sections.

#### 4.3.4.1.1 Fewer Leaks

The fundamental goal of EPA's hazardous waste management regulations is the protection of human health and the environment. The strategy for the achievement of this goal has been to set a 'no migration' lining system goal for land disposal units. The EPA recognizes, however, that this 'no migration' goal is not always achievable. Through the EPA's 'liquids management strategy' and through the use of BDAT for double liner systems, it is believed that waste management units incorporating double liner systems can come very close to the 'no migration' goal. As discussed in other Chapters, EPA's liquids management strategy has two goals: minimize leachate generation, and maximize leachate removal. Construction Quality Assurance of the earthworks and lining system installation applies directly to the second of these goals. The higher the quality of the earthworks and lining system, the closer the attainment of this goal.

It is possible, with proper design, construction, and CQA procedures, to control the leakage through a lining system and retain it within the closed system. One purpose of the CQA Program is to minimize the numbers of leaks which can occur through a given liner, and thereby maximize the security of the overall system. The detection of problems, by the careful monitoring of all installation operations, which would otherwise have gone undetected, will result, in most cases, in a lining system that meets the EPA goal of minimized leakage, thereby preventing the migration of waste materials through the system, and benefiting the health and safety of the people in the area. Simply stated, EPA believes that it is not possible to construct a liner which does not leak; however, with good design, construction, and CQA, it is possible to construct a lining system which does not measurably leak and thereby meets the EPA goal of minimizing leakage into the environment.

The consequences of a hole in a given liner have been discussed in previous sections of this document. The benefits of detecting and repairing a flaw which would become a leak are therefore apparent. There exists little published documentation, at present, which indicates the direct benefit of Construction Quality Assurance, in the context of identification of the proportion of flaws that are detected by the CQA personnel, relative to the flaws which were detected and would have been repaired by the Installer anyway. However, those data which are available clearly support the benefits of CQA. Based on experience, it is reasonable to assume that the incidence of leaks will be between one and two orders of magnitude lower in a project for which intensive quality assurance has been provided. In other words, if a flexible membrane liner, for which quality assurance of the installation was not provided, exhibits leaks at a frequency of 1 every 30 m (1 per 100 ft.) of seam, the same installation, if intensive quality assurance had been provided, would have exhibited leaks at a frequency of 1 per 300 m (1 per 1000 ft.) of seam to 1 per 3000 m (1 per 10,000 ft.) of seam.

This quantification of the benefit is important for several reasons. First, the cost of Construction Quality Assurance is high, representing an appreciable proportion of the facilities cost [Giroud and Fluet, 1986]. As with any CQA-related activity, the product which is received is not visible. The Owner/Operator must therefore understand the benefits of this cost, in that this additional capital cost can be shown to reduce future maintenance costs, and in the case of leakage resulting in decommissioning and repair, can reduce or eliminate future capital costs and costs associated with loss of use, assuming repair to be a viable alternative. Additionally, good CQA greatly diminishes the liability of all concerned parties, including the Designer, Manufacturer, Fabricator, Installer, and Owner/Operator.

Other benefits of Construction Quality Assurance are not necessarily quantifiable, but may be equally as beneficial as the monetary issue of future savings. These benefits are extremely important in the decision-making process, and so are briefly discussed in the following sections.

#### 4.3.4.1.2 High Confidence in the Integrity of the Unit

One benefit of CQA is a high confidence level by all of the parties that the system will perform as it is intended. In the early years of FML's, the industry had several failures, which resulted in a lack of confidence by some in flexible membrane liners. It is extremely likely that almost all of those failures could have been prevented by better design and/or the institution of programs of Construction Quality Assurance (including CQA of the design). Within the context of this document, CQA now includes:

- quality assurance of manufacturing, which would identify manufacturing defects, and would encourage the manufacturers to initiate very good in-plant quality control procedures;
- quality assurance of fabrication, which would detect flaws in factory seams;
- quality assurance of construction, which would detect most of the leaks which lead to failures; and
- quality assurance in the post-construction phase, which can prevent catastrophic failures by the anticipation of failure to the extent that protocols exist for the response to a potential problem, identified by the monitoring system and program.

Some failures of structures which were lined with flexible membrane liners in the early days of the industry have been documented. Construction Quality Assurance Programs, properly performed, would almost certainly have prevented these failures. There are, of course, other circumstances and mitigating factors influencing individual cases, but as a general statement, the benefit of Construction Quality Assurance Programs is that they would have prevented the overwhelming majority of those failures. Furthermore, most of the remainder of the problems would have been prevented by CQA of the design.

The high confidence level that we are therefore given by the addition of the Construction Quality Assurance Programs to the construction of these systems is a definite benefit. Confidence in the system, by all of the parties involved, will clearly assist in the provision of a better product. If, in future, CQA is extended to include the design of the lining system, the level of confidence will increase even more.

#### 4.3.4.1.3 High Confidence in the Public's Eyes

Another benefit of Construction Quality Assurance is an increase in the confidence of the public in relation to the integrity of the unit. This is always a problem, particularly in the immediate vicinity of the facility. The demonstration that the other benefits of Construction Quality Assurance are real (i.e., fewer leaks, and containment and control of leaks), will increase public confidence that the potential adverse consequences, which would affect the people directly (in particular, their health), will not arise. This public confidence in turn will be of considerable intangible benefit to both the Owner/Operators and the responsible Permitting Agencies.

#### 4.3.4.1.4 Benefits After Construction

One of the greatest tangible benefits of the provision of a Construction Quality Assurance Program is the detailed documentation of the installation, which is of considerable benefit in the diagnosis of any post-construction problems which may arise. Complete documentation should be provided to include, at the least, the following:

- documentation of all seaming activities, including ambient and apparatus temperatures, personnel identification, times, location and length of seams, etc.;
- documentation of all nondestructive testing activities;
- documentation of all sample locations for destructive testing, test results, and actions taken;
- documentation of all repairs carried out, including locations, type of repair, dates, and confirmation of nondestructive testing of the repair;

- photographic documentation of all activities; and
- preparation of record drawings for the lining system installation, illustrating the location and nature of all repairs, construction details of the system, and any components of the system which differ from the project plans and specifications (in this regard, these are not as-built drawings,,but rather a record of the sequence and activities of the construction).

### 4.3.4.1.5 Benefits to the Owner/Operator

The benefits of a thorough, well-documented CQA Program are considerable. Notwithstanding the regulatory requirements and benefits, in the context of the Owner/Operator, these benefits can be described as follows:

- greater assurance that the lining system will perform as designed;
- reduced costs of maintenance or remedial work during operations;
- reduced cost of management of the unit, due to continued satisfactory performance;
- reduced amortized maintenance cost over the lifetime of the unit;
- reduced time required for issuance of operational permits since regulators have an increased level of confidence; and
- reduced liability.

#### 4.3.4.1.6 Benefits to the Other Parties

In the context of the other parties responsible for the supply of the materials and services, these benefits can be described as follows:

- greater assurance that the products meet the specified requirements, which reduces warranty cost for the Manufacturers, but also enhances the image and reputation of the Manufacturers, ensuring long term growth and prosperity;

- lower production and installation costs due to less rework, repair, replacement, etc.;

- better productivity for all parties;

- reduced risk of litigation for all parties, resulting in reduced legal costs;

- better communications between all parties, which helps intercept potential problems on a timely basis; and

- better marketability of the products of the Manufacturers, and of the services of the Designer, due to the better quality of the product.

## 4.4 SCOPE OF CONSTRUCTION QUALITY ASSURANCE PROGRAM

The proposed Liner/Leak Detection Rule currently addresses only Construction Quality Assurance. However, we believe that the CQA process should extend back to the pre-construction design stage of a project.

### 4.4.1 Pre-Construction Stage

The pre-construction phase of the CQA Program includes all of the activities which will affect, directly or indirectly, the construction of the unit. Consequently, although not usually perceived to be components of a project which would be subject to review from the CQA personnel, this is a very important part of the program, directed toward anticipation and prevention of situations which could present problems during construction and operation.

#### 4.4.1.1 Design

The design of the unit should undergo considerable QA review. The design should be peer reviewed, and the optimum source for this review is the CQA Officer, since he/she is responsible for the quality of implementation. For this reason, the CQA Officer must be qualified and experienced in the design of composite soils/geosynthetic lining systems, as well as the soils components of the structures. This peer review must be carried out in detail, and include the review of all design calculations for soils and geosynthetics, such as sizing of the drainage layers to accommodate the anticipated volume of leachate, soils factors such as stability considerations, and the final plans and specifications.

In addition, at this stage of the pre-construction activities, the detailed, site-specific Construction Quality Assurance Plan (Section 4.2) should be prepared by the CQA Contractor or the Designer for both soils and geosynthetic components of the system. Whoever (CQA Contractor or Designer) prepares the CQA Plan, the other should provide QA peer review.

#### 4.4.1.2 Materials Specifications

The CQA Contractor should review the design specifications for the lining system materials, including both the soils and the geosynthetics. This process should include the testing and evaluation of the candidate materials on the basis of chemical compatibility with the specific contained wastes, as well as the mechanical and/or hydraulic characteristics required to satisfy the design, and the constructability of the materials.

Soils materials specifications should be based on the geotechnical evaluations for the site, and the testing which has been performed on the on-site and borrow soils for the system. Peer review of the testing and evaluation of soil properties can be carried out at this stage.

The final specifications for the soils and geosynthetic materials can be conducted as part of this task, if not previously performed in conjunction with the peer review of the final design.

#### 4.4.1.3 Materials Procurement

The Manufacturer's quality control documentation which must be submitted with the bids should be reviewed by the CQA Contractor to ensure conformance with the materials specifications.

The Independent Testing Laboratory should be selected by the CQA Contractor, on the basis of quality of work, experience, capabilities, and data turnaround.

### 4.4.2 Construction Stage

#### 4.4.2.1 Site Preparation and Foundations

The site preparation includes all of the earthworks required to develop the unit. No geosynthetics installation is conducted in this particular phase, unless a geosynthetic pressure relief system is installed. Construction Quality Assurance of the site grading and development is necessary, including the subgrade preparation, earthwork operations, and temporary roads or structures required for construction purposes. Important steps in subgrade preparation to ensure a structurally stable foundation should include excavation, placement, and compaction of soil lifts, embankment and slope construction, surface smoothing and soil sterilization.

Quality control testing, consisting in part of the selection of samples for laboratory testing for the development of compaction standards (see Section 4.5.1.1.1) is required for all soils placement at the site. In addition, nondestructive in place density and moisture content tests should be carried out by the CQA Contractor to monitor the acceptability of the soils as placed, and to document the tests so that there is an accurate record of the operations, recorded as they are being undertaken.

In conjunction with the monitoring and documentation of the soils placement and compaction operations, the CQA Contractor should also collect undisturbed samples from compacted fills for laboratory testing to ensure conformance with the plans and specifications.

#### 4.4.2.2 Dikes

A dike in a waste management unit functions as a hydraulic barrier as well as a retaining structure, resisting the lateral forces of the wastes, liners, and leachate collection and removal systems. A dike is also the above-ground extension of the foundation, providing support to the unit components above. In addition, dikes can be used to create separate cells for different wastes within a large landfill, surface impoundment, or waste pile unit. Dikes, therefore, must be designed, constructed and maintained with sufficient structural stability to prevent failure.

The CQA Program for dikes can be used to ensure that completed dikes meet or exceed design criteria, plans, and specifications. These activities may include examining the prepared dike foundation, monitoring fill materials, placement and compaction, construction of drainage systems and implementing erosion control measures.

Materials to be used for dike construction should be monitored to confirm that they are the same as those specified by the design and that they are uniform, so that no unsuitable materials are included in the dike. A test fill may be considered to verify that the specified fill dry density, moisture content, and strength can be obtained with reasonable compactive effort.

#### 4.4.2.3 Compacted Soil Liners

The compacted soil component(s) of the lining system is one of the most important parts of the soils Construction Quality Assurance Program. As part of the CQA Program a Test Fill should be required and the in-place hydraulic conductivity of the Test Fill should be measured. The Test Fill should be constructed using the same borrow soil, compaction equipment, and construction procedures as proposed in the unit. A test fill should be required because laboratory hydraulic conductivity tests frequently overestimate the actual field hydraulic conductivity [US EPA, 1987]. A field hydraulic conductivity test on the Test Fill is necessary to confirm that the materials and procedures used in the field will result in a compacted soil liner with a hydraulic conductivity of $1 \times 10^{-9}$ m/s ($1 \times 10^{-7}$ cm/s) or lower. The Test Fill program will establish the optimum methods and procedures (e.g., number of passes, moisture conditioning required) to follow to meet the minimum hydraulic conductivity criterion.

The CQA Contractor will provide the Construction Quality Assurance of the placement and compaction of compacted soil liners, including sampling for laboratory testing, and nondestructive field density and moisture content tests required to monitor the acceptability of the soils as-placed, and to document the tests so that there is an accurate record of the operations, recorded as they are being undertaken.

The acceptance criterion for the compacted soil component of the lining system is a saturated hydraulic conductivity criterion (i.e., $< 10^{-9}$ m/s ($10^{-7}$ cm/sec)). Until the laboratory and/or field testing program provides results which satisfy this criterion, the compacted soil liner component cannot be accepted. The CQA Contractor is responsible for the monitoring of all testing carried out on or off the site, and must document the results of this testing, as well as all measures taken to rectify non-conforming test results. As previously discussed (Section 4.3.1.1.9), laboratory testing of low-permeability soil samples for permeability characteristics is sensitive to sample disturbance and simulation of field stress conditions. The CQA Contractor must ensure that the collection, handling, and transportation of these samples is carried out in such a manner that the disturbance of the samples is minimized, and that the results which are obtained are as representative as possible of the field permeability of the compacted soil layer.

### 4.4.2.4 FMLs and Other Geosynthetics

The Construction Quality Assurance of the geosynthetics components of the lining system includes the monitoring of all activities by the Installer's personnel and CQA Monitors. This is absolutely essential because there are potential sources of problems at every stage of handling, deployment, seaming, nondestructive testing, and destructive testing.

Direct monitoring of these tasks involves the observation of the activity as it is carried out, recording the relevant data, and proper documentation that the activity was consistent with the plans and specifications, as well as good construction practice in general. The following specific tasks must be monitored, at the least.

#### 4.4.2.4.1 Delivery and Conformance Checking

As the materials arrive at the site and are unloaded, the CQA personnel are responsible for noting that the handling of the materials did not appear to damage them in any way, and document any damage which may occur due to accident (e.g., dropping a roll) or poor practice (e.g., dragging a roll to move it). It is the responsibility of the CQA personnel to check the documentation that arrives with the geosynthetics materials (i.e., packing notices, roll labels and identification) to ensure that the correct materials have been received (e.g., correct gauge thickness shown on the flexible membrane liner roll label). Lastly, it is the responsibility of the CQA Contractor to collect samples for conformance testing, at a frequency specified in the specifications and/or Quality Assurance Plan for the site. The CQA personnel are also responsible for the packing and shipping of these samples to the Independent Test Laboratory, with appropriate instructions. Conformance testing by the Independent Test Laboratory should include physical properties and testing to confirm that the polymer and additives used to manufacture the sheet are as specified.

#### 4.4.2.4.2 Deployment and Visual Examination

The CQA Monitor should first observe and test the compacted soil liner or other material upon which the FML or other geosynthetic will be placed to confirm that the soil or material has been properly compacted and is smooth and ready for placement of the FML or other geosynthetic. When the rolls or panels of FML or other geosynthetics are deployed (i.e., unrolled), the CQA personnel should undertake a visual examination of the entire surface of the roll or panel, to confirm the orientation and location of that roll or panel, and to detect any flaws or damage. Any flaws or damage which are encountered should be appropriately marked for repairs, and it is the responsibility of the CQA Contractor to ensure that they are noted, documented, and that the appropriate parties are informed. Documentation of subsequent repair, testing, and acceptance based on the test results is carried out by the CQA personnel. Where excessive damage is noted, a meeting of the Owner/Operator, CQA Officer, and Installer is held to determine whether or not the roll or panel is repairable. If the decision to reject the panel is made, a CQA Monitor should record it and confirm that the roll or panel is taken up and removed from the site. If portions of a panel can be salvaged for use elsewhere, their acceptability must be confirmed by the Designer.

#### 4.4.2.4.3 Seaming and Joining

Each seaming crew at the site should be accompanied by a CQA Monitor. This is of primary importance with the flexible membrane liner seaming, as the integrity of these seams is the most critical to the successful performance of the lining system. In general, geonets and geogrids do not require stringent joining criteria (since the design should preclude tension at seams), but adequate overlaps or ties should be monitored and confirmed prior to the overplacement of the next layer of materials. Geotextile filters should be sewn, and destructive tests performed in accordance with the requirements of the plans and specifications. In any event, the CQA Contractor should confirm that all geotextile seams that require sewing have been completed.

In the specific case of flexible membrane liners, the CQA Contractor should monitor a trial seam for each piece of seaming apparatus in use at the site, as well as for each equipment operator. This trial seam (carried out on a sacrificial or waste piece of liner) should be field tested in shear and peel (see Section 4.5.2.2), and should pass that testing in accordance with the acceptance criteria established in the lining system specifications. If this trial seam fails in either shear or peel, the equipment should be adjusted appropriately, and another trial seam prepared. No piece of seaming apparatus, and no seaming operators should be allowed to prepare any flexible membrane liner production seams before passing a trial seam test. These trial seam tests should be conducted at the beginning of every production shift, and other times as deemed necessary by the CQA personnel (e.g., a change in temperature or weather during a shift). The CQA Monitor should record all of the relevant apparatus settings, weather conditions, name of the seamer, and the pass or fail result for every trial seam carried out at the site.

#### 4.4.2.4.4 Nondestructive Testing of Seams

All flexible membrane liner seams should be nondestructively tested over their entire length, using appropriate industry-accepted procedures, as outlined in the Construction Quality Assurance Plan for the site. These procedures may include, but are not necessarily limited to visual observation, probe testing, vacuum testing, spark testing, air lance testing, and ultrasonic testing. Other test procedures may become appropriate as new technology develops and is proven to be effective. The CQA Contractor should monitor the nondestructive testing of every crew undertaking this activity at the site. All seams failing the test should be marked by the CQA Contractor, including an estimate of the extent along the seam of the inadequate section.

The CQA Contractor should document that the failed area is repaired by the Installer, and retested with the nondestructive test apparatus, and that this retesting results in a passed test. The location, date, type of repair, and retest should be properly documented by the CQA Contractor, along with other pertinent weather data, apparatus identification, and seamer data which are necessary for the proper identification of the location and confirmation of the repair.

### 4.4.2.4.5 Destructive Testing

Test strips should be removed from the flexible membrane liner seams at locations determined by the CQA Contractor, on the basis of unusual conditions, suspicious seam quality, or at predetermined intervals at a frequency which meets or exceeds the specified minimum frequency. These seams should be destructively field tested on site, and based on whether the seams pass or fail, appropriate action taken. The Construction Quality Assurance Plan for the site should outline the protocol for the follow-up of failed on-site destructive tests, and failed areas should be investigated and repaired in accordance with these procedures. The CQA Contractor should ensure that the sampled locations are repaired and nondestructively retested. This follow-up process should be thoroughly documented, including location, test result, sample number, etc.

In addition, the CQA Contractor should be responsible for the selection of seam sample location, seam sample collection and sample shipment to the Independent Test Laboratory for testing. These samples should be of sufficient size to yield the required number of test specimens, and to allow division of the sample into three: one for the Independent Test Laboratory, one for the Installer's test laboratory (if he has one), and an archive sample to be retained by the Owner/Operator. The sampling frequency for this testing should be specified in the Construction Quality Assurance Plan, but should normally be at a specified minimum frequency (such as one test for every 150 m (500 ft.) of seam), or more frequently, as dictated by site conditions, anticipated problems, etc.

### 4.4.2.4.6 Other Considerations

The CQA Contractor should be responsible for the examination of each prepared surface prior to the placement of the next layer of the geosynthetic or soil. This should include an examination of the compacted soil subgrade surface prior to the placement of the first layer of geosynthetics, and similarly for each successive layer of soil or geosynthetics comprising the lining system. Continuous CQA monitoring is suggested during placement of soil cover or other materials over FML components of the lining system. Any observed damage to the FML during cover placement should be expeditiously repaired.

In the case of the compacted soil subgrade, the Installer should submit documentation accepting the condition of the subgrade, at which time he accepts responsibility for ensuring that the subgrade condition is preserved up to the point at which it is covered. The CQA Contractor should ensure that this documentation is presented in the final report of the Construction Quality Assurance Program.

The proper functioning of the geosynthetic components of the system is predicated upon their installation in the manner prescribed in the specifications. As a result, the CQA Contractor should ensure that the geosynthetics are clean prior to being covered. Any dust, debris, or accumulated dirt due to operations or other contamination occasioned by rainfall, erosion of the soil components, or uncovered portions must be removed prior to covering the contaminated layers or the material must be replaced. The geonet drains and geotextile filters are particularly sensitive to this type of problem, and are the most likely to be adversely affected, if measures to clean them are not taken. In some cases, it may be necessary to remove one or more in-place layers of geosynthetics, in order to gain access to the contaminated geosynthetic, particularly when the edge of several layers is exposed and damaged by washing soil particles into the exposed edge of the system. If the contaminated layers cannot be cleaned, they must be replaced.

All of these activities should be documented by the CQA Contractor in his daily reports.

### 4.4.2.5 Leachate Collection and Removal Systems

The CQA Program for leachate collection and removal systems must provide reliance that the installed system meets or exceeds the design specifications. The functions of a leachate collection and removal system above the top liner (LCRS) in a double-lined landfill or waste pile unit are to minimize leachate head on the top liner and to collect and remove liquids from the unit, during the active life and post-closure care period. The purpose of a leachate detection, collection and removal system between the two liners (LDCRS) of a double-lined waste unit is to rapidly collect and remove liquids entering the system, also through the post-closure care period. By providing for rapid leachate removal, the LDCRS will greatly minimize the hydraulic head on the bottom liner and, thereby, minimize or eliminate leachate migration out of the unit. As the LDCRS will also be used to detect leaks in the top liner, the CQA Program must ensure that the system is installed as designed for that purpose by meeting the detection sensitivity and detection time performance standards discussed in Chapter 2 of this report.

Observing and testing the subcomponent materials of the leachate collection and removal systems as they are delivered to the site and installed are necessary to confirm and document that these materials conform to the design criteria, plans, and specifications. This observation and testing applies to the granular materials, geosynthetic materials, piping and sumps, and any other materials that make up a leachate collection and removal system.

Below are summaries of key factors that need to be addressed while constructing the LCRS and LDCRS. Two potential problems related to installation are (1) damage to the collection system during installation resulting from excessive stress and (2) leachate flow obstruction through the system. A third potential problem is contamination of geosynthetic components of the leachate collection and removal system by dust, debris or other materials. This was discussed in Section 4.4.2.4.6 and is not repeated here.

### 4.4.2.5.1 Leachate Collection Pipes

Leachate collection pipes installed in trenches at the base of a landfill or waste pile and between the liners in a landfill, surface impoundment, or waste piles are subjected to loads from construction equipment during installation, operation activities during the active life, and the waste itself. In a well-designed trench, only a small fraction of the load of a wheel or tracked vehicle applied at the top of the trench should be transmitted through the trench backfill to the pipe. However, the percentage of the load transmitted increases rapidly as the vertical distance between the loaded surface and the top of the pipe decreases. In addition, moving loads cause impact loading, which is generally considered to have a one and one half to two times the effect of stationary loading. Thus, backfill procedures and equipment traffic over pipe trenches must be monitored carefully to prevent damage to pipes.

### 4.4.2.5.2 Obstructions to Leachate Flow

The second consideration when installing a leachate collection and removal system is to provide confidence that the flow of leachate through the system is not impaired by construction activities or occurrences. Collection systems generally are designed so that leachate generated within the unit drains first through a soil or geosynthetic filter before entering the drainage layer. The purpose of this filter is to remove any fine particles that otherwise would clog the drainage layer and prevent its functioning. The filter, therefore, must be designed and constructed carefully to perform under the expected conditions. The leachate then flows through the drainage layer, which comprises permeable soils or geosynthetic drainage materials placed over the liner. If this layer does not have sufficient transmissivity (thickness times hydraulic conductivity) to accommodate the maximum leachate flow, the flow will be held up, and hydraulic head will build up on the liner. Achieving the designed thickness can be made more difficult by improper installation procedures, such as placing a granular drainage layer during high wind or intense rain, which may displace the soil so that it is no longer of uniform thickness. Another weather-related problem is drainage material contamination with fine soil particles, which decreases the drainage layer's hydraulic conductivity. This can occur as a result of soil particle erosion into granular or geosynthetic drainage layers from runoff from facility side slopes, mud, or windblown dust. These types of problems can be minimized by monitoring and testing activities that check the critical factors in the leachate collection system.

Installation procedures must be monitored to confirm that granular soils used for the drainage layer meet design specifications for size distribution of particles. In particular, excessively fine soils must not be allowed, because they will decrease the hydraulic conductivity of the layer and will clog collection pipes. On-site washing of the granular soils to remove fines may be necessary to achieve the required properties. Similarly, geosynthetic materials must be conformance tested to ensure that they meet design specifications, and they also must be covered to keep them clean.

### 4.4.2.6 Final Cover Systems

The successful construction of the final cover, like the other unit components, relies on following recommended practices for construction, employing experienced personnel, and conducting a CQA Program. The CQA Program for final covers at all land disposal units must provide assurance that (1) all layers of the final cover are monitored for uniformity, imperfections, and damage; (2) the materials for each layer are as specified in the design specifications; and (3) each layer is installed or constructed to meet the design requirements.

The following is a summary of the key factors that should be addressed for final cover construction cover at landfills, surface impoundments, and land treatment units.

#### 4.4.2.6.1 Subsidence

Subsidence under a final cover may cause problems similar to those experienced when the subgrade under a liner subsides. A flexible membrane liner may fail in tension if the waste that comprises its subgrade subsides differentially. If the final cover uses a layer of compacted soil, the soil layer may develop cracks as a result of differential subsidence that allows rainwater to infiltrate. In addition, differential subsidence may result in rainwater ponding above the final cover. The ponded rainwater may have an increased chance of penetrating the cover even if the soil is intact because of the increased pressure head on the liner. If a cover of any type has failed, ponding prevents runoff from leaving the area and provides additional opportunities for leachate production.

For covers, the problem of subgrade subsidence begins with waste placement. The waste may not have sufficient bearing strength to support the weight of additional waste and soil cover material placed above it. In addition, if the waste is not well compacted and placed so that void spaces are filled, proper compaction of the liner bedding material will not be sufficient to prevent subsidence. Therefore, to minimize subsidence, waste placement must be considered a part of final cover subgrade preparation. Cover subsidence resulting from improper waste compaction may be less of a problem today than it has been in the past. Wastes were not compacted well or at all in older landfills or disposal surface impoundments when problems associated with final cover subsidence were not well known. Now, however, virtually all landfills compact their waste. Nonetheless, differential settlement because of waste subsidence continues to be a serious problem that must be anticipated in the cover system design. Some key considerations in the design of cover systems include:

(i) The stress-strain properties of the cover system FML, geosynthetics and soils;

(ii) the ability to maintain minimum slopes for gravity drain systems;

(iii) the slope stability of layers above FML's and geosynthetics;

(iv) the use of subgrade reinforcement of stabilization methods, such as geosynthetic reinforcement or dynamic compaction to improve stability.

### 4.4.2.6.2 Installation Procedures

The construction process for final covers at landfills and disposal surface impoundments involves subcomponents similar to many of the components previously discussed, such as foundations, compacted low-permeability soil liners, flexible membrane liners, and drainage layers (leachate collection and removal systems). There is too little documented information to substantiate the quality of final covers that are constructed to comply with the landfill and surface inmpoundment requirements in Parts 264 or 265. However, it is believed that most of the installation problems for final covers for these units should be similar to those experienced installing liners, dikes, and leachate collection and removal systems.

For example, the compacted low-permeability soil layer and FML in a final cover is constructed much like the low-permeability soil and FML liner. However, the foundation for the final cover may have a lower bearing strength than the soil liner foundation; this may require using different construction techniques to achieve the required compacted soil hydraulic conductivity in the field. Additionally, the design may specify foundation (waste) soil reinforcement and such soil reinforcement must be carefully monitored during installation by Construction Quality Assurance personnel. As with the compacted low-permeability soil and FML liner, it is necessary to monitor the construction of the compacted low-permeability soil and FML cover layer.

Installation procedures for FMLs in a final cover include proper on-site storage, handling and placing of the panels to ensure proper positioning, allowing enough slack in the material for it to fit around angles and penetrations, proper seaming and anchoring procedures, and installation only during proper weather conditions.

#### 4.4.2.6.3 Vegetative Layers

The key factors that need to be addressed for constructing the vegetative layer of the final cover at land disposal units include: vegetative layer soil quality and thickness, seeding uniformity and timing, and vegetation establishment. The vegetative layer is the only layer of the final cover required for properly operated land treatment units under a permit.

Vegetation establishment and maintenance can be accomplished only by carefully addressing the soil type and the nutrient and pH levels to provide the proper soil conditions for successful seed germination and vigorous growth. The thickness of the vegetative soil layer also must be as specified in the design to provide proper root development and a sufficient moisture reserve to sustain the vegetation during dry periods.

The timing of the seeding is probably the most important factor in successfully establishing a vegetative cover. The timing will depend on whether the plant species selected is a cool- or warm-season species and on local climate conditions. The recommendations of the locan country agricultural extension agent or seed company should be used. The CQA Plan must address seeding procedures so that the recommendations are followed.

### 4.4.3 Post-Construction Stage

#### 4.4.3.1 Reporting

The CQA Contractor should be responsible for the preparation of a final report on the construction of the land disposal unit. This report should contain, at the least, all of the activities identified herein, properly documented to allow the retrieval and interpretation of the information. This includes, but is not necessarily limited to the following:

- detailed documentation of the laboratory testing carried out on the soils used for earthworks, the granular components of drainage and leachate collection and removal systems, and the compacted soil components of the lining system;

- detailed documentation of the Test Fill program, including all iterations and modifications utilized to develop the final criteria for attainment of the specified compaction and/or hydraulic conductivity requirements in the earthworks and compacted soil components of the lining system;

- detailed documentation of the field testing to determine as-placed densities and moisture contents for earthworks and compacted soil liners (this should include every test result obtained in this program, identifying the equipment used to conduct the test, the operator, the location and depth of the test, and a cross-reference in cases where retesting was required);

- detailed documentation of all of the rolls and panels of geosynthetics supplied to the site, with appropriate identification;

- detailed documentation of all of the conformance testing carried out on the geosynthetic materials supplied to the site, and a cross-reference to indicate the action taken for any materials which were rejected;

- detailed documentation of every trial seam carried out for each piece of apparatus, and each operator, for every shift worked in which seaming was carried out, including failed trial seams and the retests;

- detailed documentation of the seaming operations carried out at the site, including the location of the seam, length of seam, panel numbers, operator, apparatus number, weather conditions, and time seamed;

- detailed documentation of the nondestructive testing of the seams, including the location of the seam, length of seam, panel numbers, operator, apparatus number, weather conditions, time tested, test result, and a cross reference, for failed seams, to the retest of that seam which obtained a passing result;

- detailed documentation of the field destructive test results carried out on specimens cut from the in-place seams, including pass/fail results, operator, apparatus number, location, and a cross reference to the repair and nondestructive testing results;

- detailed documentation of the location and nature of the destructive test samples collected for testing by the Independent Testing Laboratory, including seam and panel identifications;

- a final report from the Independent Testing Laboratory showing the results of all of the destructive test samples collected from the FML seams, and the results of the conformance testing conducted for all geosynthetics;

- detailed documentation of the fabricated seam quality control testing carried out in the fabrication facility whether or not this was observed by the CQA Consultant (if not, this information should constitute a required submittal of the Fabricator);

- detailed documentation from the Installer indicating his acceptance of the soil subgrade prior to the placement of any components of the lining system (if the same Installer is responsible for both the earthworks and the compacted soil liner, as well as the geosynthetics installation, then this may not be required);

- detailed photographic documentation of the activities at the site, highlighting problems encountered, actions taken, and final resolutions; and

- detailed record (as-built) drawings of the lining system, indicating the locations of all repairs, destructive sample tests, the layout of panels and/or rolls and their numbers, and all relevant details and cross sections required to obtain an accurate picture of the system as it was constructed.

The final report for the CQA Program should include a statement, signed and sealed by the responsible CQA Officer, that all of the components of the system were installed in conformance with the plans, specifications, and good construction practice. Any exceptions to this statement should be noted, and the action taken fully described. The CQA Officer should be a registered professional engineer (P.E.) in accordance with the laws of the state in which the facility is located.

### 4.4.3.2 Monitoring

Any monitoring program involving the lining system, or the leachate collection and removal system, in the form of leak detection or investigation should at least indirectly include the CQA Contractor, by virtue of his primary knowledge of the system as installed. In any event, any problems with the system that are encountered, should be investigated by a team which should include, at the least, the Designer, the Installer, the Owner/Operator, and the CQA Contractor. Any corrective construction work carried out to repair damage or other problems should be provided with full Construction Quality Assurance by the CQA Contractor, and ideally the same personnel as were involved in the original program.

### 4.4.3.3 Coupon Testing

Coupon testing is sometimes incorporated into post-construction CQA activities, although it is an optional component of the Construction Quality Assurance Program. Samples of all of the materials are collected, and after start of operations, immersed in the leachate collection and removal system sump. Subsequently, samples from the coupon may be removed and tested in the laboratory to evaluate the durability and aging characteristics of the coupon material. This may be incorporated into any unit system, for assistance in the evaluation of the performance and condition of the liner.

## 4.5 TESTING PROCEDURES

### 4.5.1 Soils

### 4.5.1.1 Procedures

The soil components of the construction activities should be tested, wherever possible, in accordance with accepted American Society for Testing and Materials (ASTM) standards. The same standards are applied to different soils, with slight variations, regardless of the purpose of those soils (e.g., the same field compaction density test is carried out on soils for the compacted soil liner as for the soil materials used to construct dikes and embankments, although the required test result may differ). The following test procedures should be utilized for the testing of soil components of the waste management unit.

#### 4.5.1.1.1 Laboratory Testing - Soil Compaction

Laboratory tests are carried out on the particular soils to be placed at the site, in order to develop the required compaction criteria. The following tests are those which are utilized for this purpose:

- Standard Proctor Test - ASTM D698 - "Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures, Using 5.5 lb (2.49 kg) Rammer and 12 in (305 mm) Drop" [ASTM 4.08]. This test is referred to as the Standard Proctor test, and determines the maximum density attainable for a given level of compactive effort in a laboratory mold. The field degree of compaction is then compared to this density, expressed as a percentage. The typical field compaction specification would therefore call for some percentage, e.g., 97 % of the Standard Proctor maximum dry density.

- Modified Proctor Test - ASTM D1557 - "Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10 lb (4.54 kg) Rammer and 18 in (457 mm) Drop" [ASTM 4.08]. This test is referred to as the Modified Proctor test, and is carried out in essentially the same manner as D698, except that the soil is compacted into the mold in thinner lifts (i.e., 5 lifts instead of 3), and greater compaction energy is used. As a result, the compaction criterion is virtually always more stringent (i.e., 97 % of Standard Proctor maximum dry density will almost always be a lower density than 97 % of Modified Proctor maximum dry density).

#### 4.5.1.1.2 Field Testing - Soils Compaction

Field density testing is today predominantly performed by one procedure (the nuclear density test), which provides values for both the field density, and the moisture content. Sometimes, however, other methods are employed in order to provide a check of the repeatability and consistency of the results.

- Nuclear Density Test - ASTM D2922 - "Density of Soil and Soil-Aggregate In Place by Nuclear Methods (Shallow Depth)" [ASTM 4.08]. This procedure measures the density and moisture content of the soil at the surface and at depths up to 250 mm (10 in.) by the transmission of gamma rays, the intensity of which are affected by the density of the medium. The soil density can therefore be computed by the calibration of the equipment to known standards. This measured density in the field is related to the specified density (a percentage of either the Standard or Modified Proctor maximum dry density). This then determines the degree of compaction, and whether or not further compactive effort is required.

- Sand Cone Density Test - ASTM D1556 - "Density of Soil In Place by the Sand Cone Method" [ASTM 4.08]. This procedure measures the in-place density of the soil. Soil is carefully excavated from a hole in the fill, and the volume of the hole determined by filling it with a pre-calibrated sand. The weight of sand used to fill the hole is measured, so the dry density can be calculated from the weight of sand, volume of the hole, and moisture content of the soil removed from the hole. This procedure is well established, having been in common use for 30 years. It is, however, somewhat time-consuming since the excavated sample must be returned to the laboratory for moisture content determination. It has value primarily as a cross reference and a check on the calibration of the nuclear densometers being used for production field testing.

### 4.5.1.1.3 Laboratory Testing - Soil Permeability

The laboratory determination of soil hydraulic conductivity is well established within the geotechnical engineering profession. However due to the degree of sophistication of some of the equipment that is required, not all of the procedures are available to laboratories outside of the university or research environment. There are, however, two well established tests for the laboratory determination of the hydraulic conductivity of soils that can be carried out in any of four pieces of apparatus; three of these four are relatively 'standard' geotechnical laboratory apparatus. It should be noted that not all of these have established ASTM test procedures. In addition, as discussed previously, the relevance of the test result to the field condition of the soils is, in part, a function of the disturbance caused during sampling, handling, transportation to the laboratory, and sample preparation, as well as by the presence of nonuniformities, secondary structures, etc., in the field.

- Falling Head Permeability Test - This test is commonly used for the determination of the hydraulic conductivity of fine-grained soils, notably silts and clays, which exhibit intermediate to low values of hydraulic conductivity (i.e., less than $10^{-7}$ m/s ($10^{-5}$ cm/sec)). The test can be performed on trimmed samples in any of the test apparatus discussed below. Another procedure is outlined by [Head, 1982].

- Constant Head Permeability Test - This type of test is intended for use solely for granular soils (sands and gravels) containing little or no fine-grained silt or clay particles. The procedure is very similar to that of the falling head test, except that the head of water is maintained at the same level throughout the test. This is relatively easy to understand since the falling head tests, if conducted on these types of soils, would either require a very large reservoir, or the test would be of such short duration that the test result would be questionable. Other procedures also exist using different apparatus [Head, 1982].

The falling-head and constant-head permeability tests described above can be performed on representative soils in any of four well established laboratory apparatus, although there are only three apparatus that can be considered to be common laboratory equipment, and only two apparatus which are represented in ASTM procedures. These four apparatus are briefly described as follows:

- Soil Permeameter- The soil permeameter cell is described in ASTM D2434 - "Permeability of Granular Soils (Constant Head)" [ASTM 4.08]. This test can be performed fairly easily on coarse grained soils with moderate to high permeability (hydraulic conductivity). It consists of a plastic cylinder which is filled with the soil, and through which water is allowed to flow. Maintaining the constant head of water on the sample, the flow through the sample is directly measured, from which the hydraulic conductivity of the soil can be calculated using Darcy's equation. Falling head tests are carried out in the same apparatus on fine-grained soils, with similar procedure except that the head is allowed to draw down as the water flows through the sample. Both of these tests are also described in [Head, 1982].

- Oedometer Cell - ASTM D2435 - "One-Dimensional Consolidation Properties of Soils" [ASTM 4.08]. The hydraulic conductivity of a fine-grained soil can be determined for any load increment during the performance of a standard oedometer consolidation test. If the soil consolidation properties are not of specific interest, the permeability test can still be performed in this apparatus by following the procedure cited for the consolidation test. This is a standard piece of geotechnical laboratory apparatus, and in fact, if the test is monitored by a data acquisition system, then the values for the hydraulic conductivity of the sample under each increment of load can be obtained directly. In the context of waste management units, this can provide an indication of the response of the soil hydraulic conductivity under load. This is a feature that is not readily available with the Permeameter. The drawback of the Oedometer, however, is that the sample size is frequently far from ideal, and the lack of representativeness to site conditions is likely maximized.

- Triaxial Cell - There is no standardized test procedure for this apparatus for determining soil hydraulic conductivity, although several different procedures are contained in the literature [Head 1986]. In this reference, three procedures are outlined which accommodate the variability in the type of equipment which may be available in a given laboratory. In the test, the flow of water through the sample is upwards, which distinguishes it from the other tests, which are gravity-driven. This does, however, allow the control of the gradient across the sample, and provides a high reliability factor to the test result. One of the features of this apparatus is that both constant head and falling head tests can be performed, which provides considerable flexibility in the sense that the same apparatus can be utilized for all types of soils. In addition, the flexible containment of the sample more closely models the field overburden conditions, which can be taken to very high levels easily with this system.

- Rowe Cell - This is another test which has not been standardized, but which is familiar to research and university laboratories. The apparatus is a relatively large-scale consolidation cell (up to 250 mm (10 in.) in diameter) on which the load is imposed hydraulically through a diaphragm and pressure system (such as the standard laboratory mercury-pot system). Due to the configuration of the cell, the test can be set up to determine the hydraulic conductivity of the soil sample with drainage either vertically or horizontally. In addition to allowing the test to be carried out under known conditions of effective stress, the loads can also be cycled. In addition, both constant head and falling head tests can be performed in this apparatus. A thorough treatment of this test and its variations is contained in [Head, 1986].

#### 4.5.1.1.4 Field Testing - Soil Permeability

The least developed component of soils testing related to the design and construction of land disposal units is the field determination of soil hydraulic conductivity. The test procedures currently in use have a response time occasioned by the low hydraulic conductivity of the soil, which results in the test taking a very long time to perform. Hence, the existing tests are primarily of value as design tests. Due to the duration, they are presently of little value as a quality control test.

For example, the sealed double ring infiltrometer (SDRI) developed by Daniel [1985] is based in part on ASTM D3385 "Infiltration Rate of Soils in Field Using Double-Ring Infiltrometers" [ASTM, 4.08]. The Daniel apparatus has been developed to overcome the constraint imposed that the test as written is not suitable for use in clay soils. This modified procedure achieves results which are reported to be more representative of field conditions over laboratory test procedures. The SDRI test tends to give a higher (up to two orders of magnitude) value for hydraulic conductivity than do those obtained from laboratory hydraulic conductivity tests. One shortcoming of the procedure is that the overburden conditions experienced in a landfill waste pile or surface impoundment are not modeled. In landfills, for instance, overburden stresses on the order of 500 kPa (10,500 psf) could be exerted, which would tend to alter the hydraulic conductivity from that measured in the absence of this condition.

The SDRI procedure reported by Daniel is, at present, the most thoroughly documented, and is currently being used for in-place hydraulic conductivity testing of Test Fills (see Section 4.5.1.4). Other procedures are being developed as well [Reynolds and Elrick, 1985a, 1985b, 1986]. In the context of CQA, however, it is necessary to rely on the care and skill of the persons sampling and handling the laboratory hydraulic conductivity tests, to provide accurate results. The extrapolation of these results to the field situation will remain an area requiring considerable experience and interpretation on the part of the Designer and CQA Contractor.

### 4.5.1.2 Effectiveness of the Tests - Acceptance Criteria

#### 4.5.1.2.1 Compaction-Related Tests

The compaction-related tests for soils are very well established, and even the most recent of these tests (D2922 "Density of Soil and Soil-Aggregate In Place By Nuclear Methods (Shallow Depth)") has been an ASTM standard test method since 1971. It is generally accepted that the reliability of the results has been proven, and this equipment is now almost universally used for this purpose.

The Modified Proctor Test (D1557) is used for a compaction standard, primarily when a very high degree of compaction is necessary for a high strength fill (e.g., dam construction). For the purposes of waste management projects, for the general earthworks other than the soils for the lining system, it is believed that the less restrictive Standard Proctor Test (D698) is more appropriate. The acceptance criteria for compaction are a function of the nature of the particular component of the structure. The general earthworks and fills would normally be compacted to a degree between 95 percent and 98 percent of the Standard Proctor maximum dry density for that soil, and densities falling below the 95 percent level are not desirable.

The low-permeability soils comprising compacted soil components of the lining system are specified on the basis of a hydraulic conductivity criterion, and compaction to a specified degree may not be necessary, except in cases where the level of compaction, soil density, moisture content, and hydraulic conductivity have been correlated. One of the purposes of the Test Fill program [USEPA, 1986] is to allow this correlation of compactive effort and density achieved at a given moisture content, to the in-place hydraulic conductivity. Consequently, for the compacted soil lining components, the field density will likely only be used for this cross reference.

#### 4.5.1.2.2 Hydraulic Conductivity Tests

The laboratory tests to determine hydraulic conductivity are well established. The apparatus such as the Triaxial Cell and the Rowe Cell are somewhat more desirable in that they can allow the use of larger samples. Regardless of the particular test selected, care must be taken at all stages from sample collection through performance of the test, to avoid the potential problem of sample disturbance. In addition, there is concern with regard to the representativeness of the sample to the 'macro-scale' field condition, whereby the secondary structure of the soil, and other discontinuities may not be modeled into the test because of the relatively small sample tested. The interpretation of the engineer carrying out the test therefore becomes an important factor in assessing the results.

### 4.5.1.3 Current State of Practice

The test procedures outlined herein and recommended for use in the Construction Quality Assurance Program represent the current state of practice. A useful tool for providing field hydraulic conductivity measurements with fast turn-around of results is still required, however, in order to be of value for Construction Quality Assurance Programs. This state of practice is underdeveloped at present. As new methods and procedures are developed, they will require considerable field testing before they can be relied on for CQA purposes.

### 4.5.1.4 Test Fills

Although Test Fills may be constructed for the unit in order to evaluate the compactability of the soils over a range of moisture contents, the Test Fill program is primarily a means of evaluating the hydraulic conductivity of the compacted soil components of the lining system.

A field hydraulic conductivity test of the compacted soil in the Test Fill should be performed to confirm that the materials and procedures used in the field will result in a compacted soil liner with a hydraulic conductivity of $10^{-9}$ m/s ($10^{-7}$ cm/sec) or lower. Field testing is not intended to preclude the use of laboratory testing in the design or construction stage or as a means of evaluating liner/leachate compatibility. The design stage and the Construction Quality Assurance Program will be expected to include a mixture of both field and laboratory hydraulic conductivity tests.

As appropriate methods are developed and verified, it will be desirable to conduct field hydraulic conductivity tests on the entire unit. Until that time, field hydraulic conductivity tests can be performed in the Test Fill without causing delays during construction of the entire unit. The field test used in the Test Fill should be performed for an adequate length of time to achieve "stable" results and to verify that the hydraulic conductivity of the compacted soil liner is $10^{-9}$ m/s ($10^{-7}$ cm/sec) or lower.

In addition to being used for a site for the field hydraulic conductivity test, the Test Fill should also verify other elements of the design and construction of the soil liner. The Test Fill construction will allow the Construction Quality Assurance Monitors to verify that equipment and construction procedures for breaking up clods, moisture conditioning (wetting and/or drying), and compacting the soil are adequate to meet the specified density, moisture content, and permeability criteria. In addition, construction monitoring activities, including measurement of lift thickness and compaction equipment coverages, can be correlated with in-place density and moisture content tests and with the field hydraulic conductivity.

EPA has published a valuable Technical Resource Document: "Design, Construction, and Evaluation of Clay Liners for Hazardous Waste Facilities" [USEPA, 1986c] that provides detailed information on construction of a compacted soil liner.

The Construction Quality Assurance Program for compacted, low-permeability soil liners must confirm that the liners meet or exceed the design specification. The purpose of a compacted, low-permeability soil liner depends on the overall liner system design. In the case of soil liners used as the lower component of a composite liner, the soil component serves as a protective bedding material for the upper component of the FML and minimizes the leakage rate through any flaws in the upper component. An objective shared by all low-permeability soil liners is, therefore, to serve as long-term, structurally stable bases for all overlying material.

Prior to construction, adequate studies should be conducted to confirm that the low-permeability soil liner design meets or exceeds regulatory requirements. These studies should include an evaluation of the proposed borrow source to confirm the existence of an adequate quantity of suitable material, with testing to determine particle size distribution, Atterberg limits, moisture/density relationships, hydraulic conductivity, liner-leachate compatibility tests, and appropriate consolidation and strength tests of fabricated samples of the proposed soil liner. This work can be replicated on the Test Fill.

### 4.5.2 Flexible Membrane Liners

#### 4.5.2.1 Procedures

Many of the procedures with which FML's are being tested were developed for polymeric materials in general, and in some cases may not be totally appropriate for flexible membrane liners. ASTM Committee D35 on Geotextiles and Related Products is working to develop FML-specific tests to supplant existing tests.

### 4.5.2.1 Laboratory Testing - Specifications Conformance

The laboratory tests for conformance are those which are carried out on samples of flexible membrane liners taken from the rolls and/or panels delivered to the site. These tests are used to verify that the FML's supplied comply with the specified properties. In some cases, for particular applications, different or additional properties may be specified for this conformance testing, but in general, the following tests will allow the evaluation of flexible membrane liner conformance with the specifications.

- Density - ASTM D792 "Test Methods for Specific Gravity and Density of Plastic by Displacement", Method A, or ASTM D1505 "Test Method for Density of Plastics by the Density-Gradient Technique" [ASTM, 8.01]. These procedures test to verify that the density of the material is within the range specified.

- Tensile Strength and Elongation - ASTM D638 "Test Method for Tensile Properties of Plastics" [ASTM, 8.01]. This test is a simple shear test carried out to record the tensile strength and elongation on break of the flexible membrane liner, to ensure that its stress-strain characteristics comply with the specifications. Requisite stress is a constant, regardless of the gauge of the material, and this can therefore be easily verified by this test, given the thickness of the material.

- Carbon Black Content - ASTM D1603 "Test Method for Carbon Black in Olefin Plastics" [ASTM, 8.02]. Carbon black content in FML's is a necessary component required to resist degradation due to ultraviolet exposure. In polyethylene flexible membrane liners, the otherwise white material is susceptible to degradation after exposure to sunlight after even relatively short periods. A minimum proportion of carbon black is required to be mixed with the resin to avoid this problem.

- Thickness - ASTM D1593 "Specification for Nonrigid Vinyl Chloride Plastic Sheeting" or ASTM D374 "Test Methods for Thickness of Solid Electrical Insulation" Method C [ASTM, 8.01]. As the mechanical properties are, at least in part, a function of the thickness of the sheet, the thickness can give an indication of whether or not there will be problems with the physical characteristics of the material.

### 4.5.2.1.2 Laboratory Testing - Destructive FML Seam Quality Control

As an important component of the overall Construction Quality Assurance Program, the quality of the flexible membrane liner field seams formed by either extrusion or fusion procedures is measured by the performance of testing to determine the integrity of the seams. These samples are collected from actual production seams at the site, and the testing carried out by an Independent Testing Laboratory, with results of the testing reported quickly, to allow action before any cover materials are placed on the liner.

- Bonded Seam Strength - ASTM D3083 "Specification for Flexible Poly(Vinyl Chloride) Plastic Sheeting for Pond, Canal, and Reservoir Lining" Modified by NSF 54 [ASTM, 4.04; NSF, 1985]. In this test, a tensile test is carried out across the FML field seam, and the conformance of the test result to the specification checked.

- Peel Adhesion - ASTM D413 "Test Methods for Rubber Property - Adhesion to Flexible Substrate" Modified by NSF 54 [ASTM, 9.01; NSF, 1985]. This is a test across the flexible membrane liner seam, in which the two pieces of the seamed rolls or sheets are pulled apart, so that the stress is exerted directly onto the seam. It is generally most appropriate for polyethylene flexible membrane liners.

### 4.5.2.1.3 Field Testing - Nondestructive FML Seam Tests

All FML seams formed in the field must be nondestructively tested over their entire length. A major component of the overall

Construction Quality Assurance Program, these techniques can be an effective means of confirming seam continuity, but are not intended to be quantitative in nature.

- Vacuum Test - No ASTM established test method presently exists for this test, although the test is widely used by Installers for the nondestructive testing of flexible membrane liner seams. In the test, a section of the seam is covered with a soapy mixture and subjected to a vacuum. Leakage is detected through the presence of bubbles, as air is drawn from below, through the seam. The test is very fast and effective at locating flaws which may not have been visually apparent.

- Air Lance Test - Another test which is not standardized, the Air Lance Test, is also used to nondestructively test flexible membrane liner seams. In this test, a high pressure air jet is trailed along the edge of the flexible membrane liner seam, and leaks detected by the ballooning of the upper liner.

- Air Pressure Test - This test is not standardized, but is frequently utilized in the case of double fusion seams of polyethylene FML's. In this test, a pressure is exerted into the gap between the two tracks, and the pressure is monitored over a brief period. A drop in the sustained pressure is indicative of a leak in the seam.

- Spark Test - In this (also non-standardized) test, a piece of copper wire is placed, so as to be embedded within the seam, after seaming. A DC current is then passed through the wire through exposed ends, and a negatively charged wand passed along the edge of the seam. Leaks will be detected by a spark jumping from the copper wire to the rod. Some problems exist with the method, such as the necessity of securing the copper wire free ends after the test. It is gaining increased usage, however.

- Ultrasonic Testing - Nondestructive testing using ultrasonic equipment is receiving considerable developmental attention at this time, although it is not presently widely accepted, and sufficient field truthing has not been carried out to date [Peggs et al, 1985; and Koerner et al, 1987]. The procedures being evaluated include both direct transmission, as well as an echo/pulse procedure. Discontinuities within the seam are indicated by variations in the oscilloscope signal received. At present, the need for considerable operator skill and interpretation is a major constraint on its use.

### 4.5.2.2 Effectiveness of the Tests - Acceptance Criteria

#### 4.5.2.2.1 Conformance Tests

These tests, as a measure of conformance of the flexible membrane liner to the specifications for the project, are generally very effective. Most of the tests were developed specifically for non-flexible membrane liner plastics, having been adopted for general purposes. It is anticipated that ASTM Committee D35 will derive and issue FML-specific tests, and these should be adopted as they become available. In the interim, the tests procedures which are being used are not deficient for testing the particular characteristics of the flexible membrane liners. As conformance tests, the attractiveness is that they are generally easy to perform. In fact, some of these tests, such as the thickness test can readily be carried out on the spot using an appropriate micrometer or other caliper.

The required specifications which must be satisfied for the conformance testing for flexible membrane liners will be project-specific, and the properties will therefore be a specific function of the design.

#### 4.5.2.2.2 Seam Quality Control Tests

The effectiveness of the current seam quality control testing is the subject of considerable discussion at present, not necessarily with specific regard to the performance or the results of the test, but rather with the interpretation of the results. This has been the topic of technical papers [Peggs et al, 1985; and Peggs, 1987], and clearly there are two issues. First, the current NSF Standard 54 states that in peel, for instance, the seam must not peel (pull apart at the fusion joint), and specifically that Film Tearing Bond must be exhibited. There are, however, a wide variety of failure patterns which can occur in this test, and no guidance exists with respect to how to determine whether or not a problem is indicated by a particular failure mode.

Another problem is more critical to the operations at the site, and the Construction Quality Assurance Program in general. When the samples are collected from the seams formed at the site, they are split in such a manner that one sample goes to the Independent Testing Laboratory, and one to the Manufacturer's (or Installer's, if a different party) laboratory. The laboratories split each sample into ten specimens, and conduct five shear tests and five peel tests per sample.

The problem is that there is no guidance with regard to the interpretation of the results. For a case in which all five peel specimens fail, for example, the conclusion is clear with regard to the test. In the more frequent circumstance, however, one specimen may fail totally, but the others pass. The question will arise if the average of the five specimens meets the specification, then is the sample a pass or a fail? At present, this must be decided at the discretion of the CQA Contractor, unless a protocol for the acceptability of flexible membrane liner destructive seam tests is developed, in which case it should be incorporated into the Construction Quality Assurance Plan. Similarly, the procedure for the evaluation of differing results from the Independent and the Installer's test laboratories should be iterated in the CQA Plan. Work is proceeding on the development of a acceptance protocol for destructive seam tests [Peggs 1987], and this problem will be resolved in due course.

### 4.5.2.2.3 Nondestructive Seam Tests

The nondestructive seam tests which are presently carried out are reasonably effective in the non-quantitative evaluation of seam continuity. They are, however, not standardized, and for that reason there is very little control over the procedure, i.e., since the test method is often not detailed, a vacuum test, for instance, can be performed that goes through an approximate procedure which satisfies the spirit of the test, but may provide a result of dubious merit. The Construction Quality Assurance Plan for the project should, therefore, outline the procedure to be followed, in the absence of a standard to reference. The CQA Contractor should then include in his responsibilities, the confirmation that the method and procedure are as contained in the CQA Plan.

Acceptance criteria for a qualitative test are of necessity vague. The purpose of the test is to provide an evaluation of the continuity of the flexible membrane liner seams. This is indicated by the lack of detection of a leak by the test. This 'evidence' of continuity must not be confused with indication of the quality of the seam, which at present can only be determined by quantitative methods (i.e., destructive testing). Finding no leaks can lead to the conclusion of seam continuity, but such is not a result in itself.

### 4.5.2.3 Current State of Practice

The testing requirements identified herein for flexible membrane liners represent the state of practice. The ASTM Committee D35 on Geotextiles and Related Products is actively pursuing new test methods prepared specifically to address the testing of flexible membrane liners. This is a slow process, however, and the current procedures will be used for some time yet. As the new tests become available, however, they should be substituted for the current practice of modifying an otherwise inappropriate test method. The lack of test procedures is not so much an indication of the negligence of the standards-writing systems, but is rather an indication of the growth of the industry and the proliferation of applications of flexible membrane liners.

It should be noted that there are many independent test methods, some of which are very applications-specific. These tests are not standardized, and in many cases, never will be. In particular circumstances, however, they may represent the state of the art, and it may be desirable to use some of these procedures for specific projects. Consequently, the designers of these facilities should remain current with the testing literature, in which these test procedures are normally well documented.

### 4.5.3 Other Geosynthetic Materials

#### 4.5.3.1 Procedures

Although the testing requirements for the various geosynthetics (other than FML's) vary from material to material, they are discussed together primarily because the same tests are normally used, and the differences lie more with the menu of tests required for each material. Discussion herein shall be limited to geotextiles, geonets, and geogrids, those being the geosynthetic materials most commonly used in these types of lining systems. As other products, such as composite products, gain acceptance for use in hazardous waste management units, they will require additional attention.

The following tests are used for the determination of the physical and mechanical characteristics of these materials in the performance of a Construction Quality Assurance Program. It should be noted that all of these tests are performed in the laboratory. At present, neither destructive nor nondestructive tests are carried out on these materials in the field.

##### 4.5.3.1.1 Laboratory Testing - Specifications Conformance

Conformance testing is carried out for geotextiles, geonets, and geogrids for compliance with the specifications, on samples taken from the rolls of material supplied to the site. The requirement for this testing will vary from one site to another, but these samples should be collected at a minimum frequency of one per 10 000 $m^2$ (100,000 ft²). The testing requirements for conformance for each of these materials are as follows, at the least.

- For geotextiles:
  - Mass Per Unit Area - ASTM D3776 "Mass Per Unit Area (Weight) of Woven Fabric" [ASTM, 7.01]. The mass per unit area is always one of the primary criteria specified for geotextiles. It is particularly important for geotextiles which are intended to function as a cushion, but also in general it is an indicator of the adequacy of the mechanical properties.
  - Grab Strength - ASTM D1682 "Breaking Load and Elongation of Textile Fabrics" Section 16, using a 4 in. x 8 in. (100 mm x 200 mm) specimen, 3 in. (75 mm) gauge length, 1 in. wide x 2 in. long grips, 12 in. (300 mm) per minute strain rate, and a Constant Rate of Extension (CRE) Machine" [ASTM, 7.01]. This test is well suited for use as a quality control/conformance test as it easy, and can be performed very quickly. Most of the strength tests for geotextiles (including this one) are borrowed from the textile test methods, and are contained in this standard.
  - Tear Strength - ASTM D4533 "Trapezoid Tearing Strength of Geotextiles" [ASTM, 4.08]. One of the few geotextile-specific test methods available at present, this test determines the tearing strength of geotextiles, propagating from an initial cut.
  - Burst Strength - ASTM D3786 "Hydraulic Bursting Strength of Knitted Goods and Nonwoven Fabrics - Diaphragm Bursting Strength Tester Method" [ASTM, 7.01]. This test is borrowed from the textile test procedures, and is commonly referred to as the Mullen Burst Test. It consists of the rupture of a specimen of geotextile by the application of a fluid load, on a membrane below the fabric. This parameter is frequently included in the specification of geotextiles.

- Puncture Strength - ASTM D3787 "Bursting Strength of Knitted Goods - Constant-Rate-of-Traverse (CRT) Ball Burst Test" Modified for Geotextiles [ASTM, 7.01]. This is a test in which a plunger with a round head is pushed through the specimen and the load recorded. This test will very shortly be replaced by a new test for puncture specific to geotextiles, currently being finalized by ASTM D35.

- Wide Strip Tensile Test - ASTM D4595 "Wide Width Strip Tensile Strength of Geotextiles" [ASTM, 4.08]. This test is normally intended more as a design test than as a quality control or conformance test. If, however, the geotextile will undergo very high tensile loadings, it should be included as a conformance test.

- Thickness - ASTM D1777 "Measuring Thickness of Textile Materials" [ASTM, 7.01]. This procedure has been slightly modified to identify the "nominal thickness" as that thickness under a compressive stress of 2 kPa (42 psf). Its use as a conformance test is frequently confined to applications in which the geotextile is to be used as either a cushion, or as a fluid transmission medium, for which the thickness is an important component of the specification.

- Normal Permeability or Permittivity - ASTM D4491 "Water Permeability of Geotextiles by Permittivity" [ASTM, 4.08]. Permittivity of geotextiles refers to the flow rate of a fluid through the plane of the fabric. Both permittivity and normal permeability are determined in this test, one being derived from the other, and having different units. This particular property is of interest primarily if the geotextile is designed to act as a filter.

- Apparent Opening Size - United States Corps of Engineers CW 02215 "Plastic Filter Fabric". This test, scheduled to be replaced by an ASTM procedure, is of value in the evaluation of geotextiles as filters for different types of soils. It determines an equivalent soil grain size for which 90 percent of the openings in the geotextile are smaller.

- For geonets:
  - Polymer Specific Gravity - ASTM D792 "Specific Gravity and Density of Plastics by Displacement" [ASTM, 8.01]. The specific gravity of the geonet is usually specified to conform to that required for the flexible membrane liner.
  - Mass Per Unit Area - ASTM D3776 "Mass Per Unit Area (Weight) of Woven Fabric" [ASTM, 7.01].
  - Thickness - ASTM D1777 "Measuring Thickness of Textile Materials" [ASTM, 7.01]. This procedure has been slightly modified to identify the "nominal thickness" as that thickness under a compressive stress of 2 kPa (42 psf).
- For geogrids:
  - Mass Per Unit Area - ASTM D3776 "Mass Per Unit Area (Weight) of Woven Fabric" [ASTM, 7.01].
  - Measurement of Spacing Between Strands - There is no test method for this requirement, which should comprise the measurement of the grid openings at several locations in the sample.
  - Wide Strip Tensile Test - ASTM D4595 "Wide Width Strip Tensile Strength of Geotextiles" [ASTM, 4.08]. This test is normally intended more as a design test than as a quality control or conformance test. For geogrids, however, the narrow strip tests are not sufficiently representative of the strength.
  - Node Strength - The only test of this nature presently in existence is a manufacturer-specific test in which a tensile force is applied to the geogrid node, rather than the rib of

the material. A specified proportion of the geogrid rib strength must be achieved through the node.

### 4.5.3.2 Effectiveness of the Tests - Acceptance Criteria

#### 4.5.3.2.1 Geotextile Conformance Tests

The conformance tests used for geotextiles are at present a mixture of textile and geotextile test methods. The applicability of some of the former is suspect because of the nature of geotextiles versus most textiles, e.g., the strengths of geotextiles are typically orders of magnitude higher than are required for textiles. As a component of the overall Construction Quality Assurance Program, these tests are effective in the context that they provide a relatively fast and easy way to confirm that the properties of the material provided to the project site conform to the specifications.

Not all of the tests cited will necessarily be required for conformance testing on a given project, and for particular situations, the design engineer may wish to add other tests. In general, some of the textile test methods are inappropriate for geotextiles. The development of standard test methods is very slow, but progress is being made. At present, ASTM Committee D35 has over 20 test methods for geotextiles in draft, and as these test procedures become available, they should supersede the tests described or referenced herein.

The acceptance criteria for geotextile conformance tests are a function of the use of the geotextile (e.g., cushion, filter) as well as the configuration of the unit in which it is to be incorporated. The properties related to the tests indicated in Section 4.5.3.1, above, are those which relate most appropriately to the normal application of geotextiles, and for which the conformance of the material to the specification is critical to performance. It is the responsibility of the Designer to write specifications for testing that clearly indicate that the requisite properties of the geotextiles have been satisfied.

#### 4.5.3.2.2 Geonet Conformance Tests

Conformance testing for geonets is more difficult in that there is only one real design criterion which is absolutely critical, that being the transmissivity of the geonet. This is a very specialized test, and is design-related more than Construction Quality Assurance related. Consequently, the most that can be done in the form of conformance testing is to confirm the materials properties, and the dimensions of the geonet, which will confirm its compatibility and stress-strain response (in particular, long term creep under constant load), as well as its flow capacity.

#### 4.5.3.2.3 Geogrid Conformance Tests

Just as only one property of a geonet is of concern (transmissivity), only one property of geogrids is of concern with regard to conformance of geogrids, that being tensile modulus (tension/strain characteristics). Confirmation of the tensile properties of geogrids requires an understanding of the manner in which tensile properties, as reported by the Manufacturers, are derived. This is more of a design concern, and the values required for the conformance tests must reflect the numbers reported or guaranteed by the Manufacturers as well as the numbers used for design.

### 4.5.3.3 Current State of Practice

The current state of practice with regard to the testing of geotextiles, geonets, and geogrids for conformance, and even in general, is that the industry is coping in the absence of a well defined set of appropriate geotextile, geonet, and geogrid standard test methods. It is more of a problem with these materials than with FMLs because the established tests for plastic sheeting, for example, are generally applicable to FMLs. Due to the differences in properties of geotextiles versus textiles, on the other hand, many textile test methods have been modified to make them usable, but often leave gaps in their usefulness. This problem is, however, decreasing, and in fact, as the ASTM D35 Committee prepares and issues a more complete set of geosynthetics-specific tests, the problem will gradually, but eventually, go away. The designers and Construction Quality Assurance Consultants must remain current on the status of the activities of Committee D35, and ensure that the properties and tests specified are current. This will be particularly important, because the Manufacturers will likely all change over their reporting and in-plant testing as soon as the new tests become available.

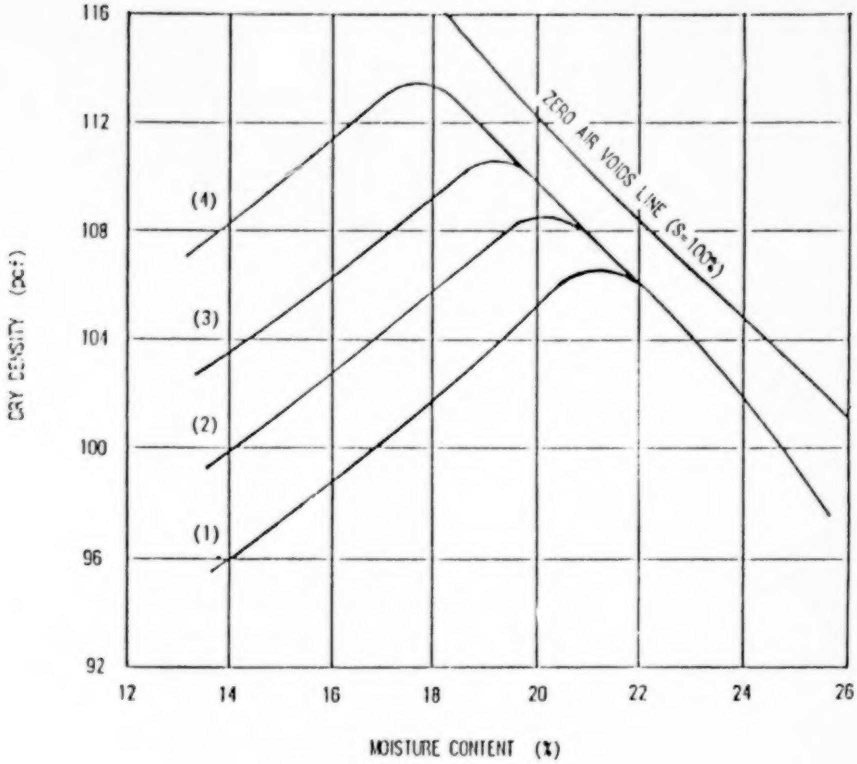


Figure 4-1. Characteristic moisture-density curves for a low-permeability soil for different levels of compactive effort (the compactive effort increases from Curve 1 to Curve 4).

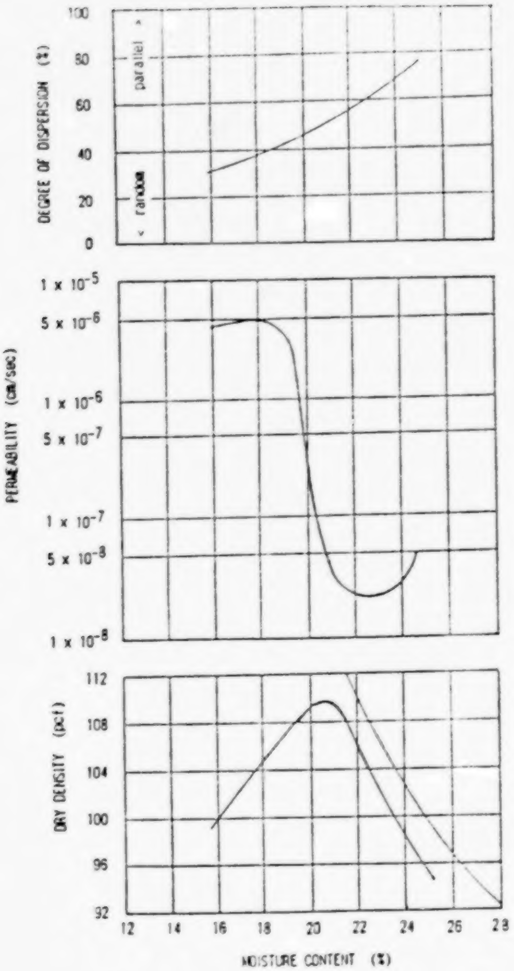
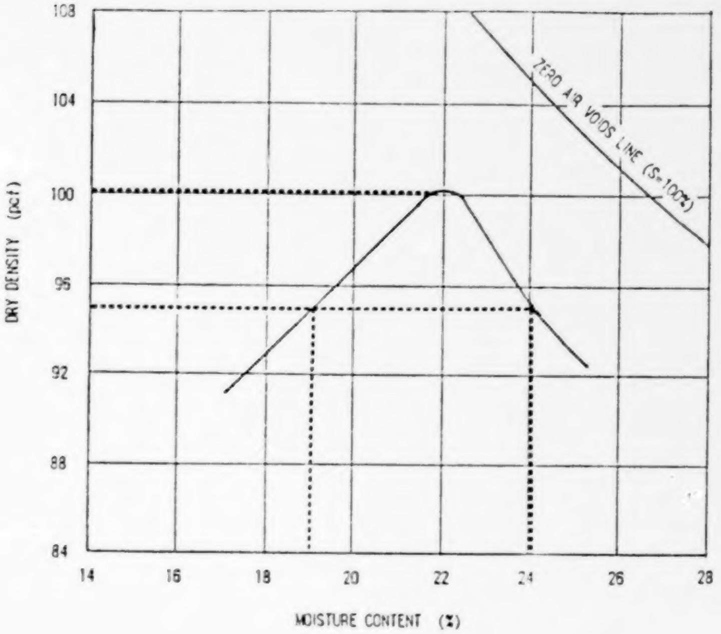


Figure 4-2. Characteristic curves for a low-permeability soil, illustrating the moisture/density, moisture/permeability, and moisture/degree of dispersion relationships at a given level of compactive effort. Adapted from [Mitchell et al, 1965; and Seed and Chan, 1959].



$\gamma_{max}$ = 100 lb/ft$^3$ $w_{opt}$ = 22 %

Required compaction = 95 % (0.95 × 100 = 95 lb/ft$^3$)

Range of moisture contents = 19 to 24 %

Figure 4-3. Moisture-density relationship: parameters pertinent to compaction specifications.

# CHAPTER 5

# LAND TREATMENT UNITS

## 5.1 INTRODUCTION

The purpose of Chapter 5 is to provide a discussion of the new standards proposed by EPA for new permitted and interim status land treatment units.

Chapter 5 is comprised of 4 sections which are summarized below.

Section 5.1 outlines the scope of this chapter.

Section 5.2 is a discussion of regulatory approaches to land treatment and EPA's proposal to expand the soil-core and soil-pore liquid monitoring plans required of land treatment units. This section addresses: the viability of land treatment; the unique challenges leak detection at land treatment units presents.

Section 5.3 is a discussion of current regulations and proposed changes to those regulations at permitted land treatment units. This section addresses: the current regulations; the approach to statutory requirements; detection confidence levels; inspection of monitoring equipment; and response action plans.

Section 5.4 is a discussion of the additional requirements proposed for interim status land treatment facilities. This section addresses: current interim status regulations; interim status implementation differences; and amendments to the interim status monitoring plan.

## 5.2 REGULATORY APPROACH TO LAND TREATMENT

Land treatment is viewed by EPA as a viable method of hazardous waste land treatment and disposal for some types of hazardous waste.

Land treatment involves applying hazardous waste on the soil surface or incorporating it into the upper layers of the soil in order to degrade, transform, or immobilize hazardous constituents. Unlike landfills, waste piles or surface impoundments, land treatment does not use double liners and leachate collection systems to contain the waste. Rather land treatment relies on the physical, chemical, and biological processes occurring in the upper layers of the soil for the degradation, transformation, and immobilization of hazardous constituents. In this sense, land treatment can be viewed as an open system.

Land treatment units depend upon a number of soil and waste interactions for success, therefore it is especially important that the unit be carefully operated and monitored. The current design and operating requirements under Parts 264 and 265 require owners or operators of land treatment units to include monitoring in the unsaturated zone to provide information that the owner or operator will use in modifying his operating practices to maximize the success of treatment processes. The principle objective of the current unsaturated zone monitoring requirements is to provide effective management of liquids at the unit to minimize the risk of groundwater contamination. At surface impoundments, waste piles, and landfills this objective is met by the double liner and leachate collection system, and the final cover that prevents liquids entering the unit and migrating into the subsoils.

The regulatory approach to land treatment, however, does seek to minimize uncontrolled migration of hazardous constituents into the environment. This is accomplished by using a defined layer of surface and subsurface soils (referred to as the "treatment zone") to degrade, transform or immobilize the hazardous constituents contained in the leachate passing through the system. These treatment processes achieve the same general objectives as the liquids management strategy used at other types of land disposal units in that they act to prevent hazardous constituents from migrating into the environment.

While the objective remains the same, the general approach must be modified somewhat for land treatment units. Land treatment units are dissimilar to other land disposal units in that they are not designed and operated to minimize all releases to ground water. On the contrary, they are open systems that freely allow liquid to move out of the unit. The goal of land treatment, therefore, is to reduce the hazardousness of waste applied in or on the soil through degradation, transformation and immobilization processes.

Two monitoring procedures, soil-core and soil-pore liquid monitoring, are required in EPA existing rules. They are intended to complement one another. Soil-core monitoring primarily will provide information on the movement of "slower-moving" hazardous constituents (such as heavy metals). Soil-pore liquid monitoring will provide essential additional data on the movement of fast-moving, highly soluble hazardous constituents that soil-core monitoring may miss.

For example, if a significant increase of a hazardous constituent is detected in unsaturated zone monitoring, the owner or operator is required under the existing Part 264 requirements to examine more closely the unit characteristics that significantly affect the mobility and persistence of that constituent. These significant unit characteristics may include treatment zone characteristics (e.g., pH, cation exchange capacity, organic matter content), or operational practices (e.g., waste application method and rate). Modifications to one or more of these characteristics may be necessary to maximize treatment of the hazardous constituent within the treatment zone and to minimize additional migration of that constituent to below the treatment zone".

## 5.3 PERMITTED LAND TREATMENT UNITS

### 5.3.1 Current Regulations

EPA's current regulations (40 CFR 264, Subpart M) for land treatment units require that the owner or operator of the unit must (among other requirements): (1) establish a land treatment program that is designed to ensure that hazardous constituents placed in or on the treatment zone are degraded, transformed, or immobilized within the treatment zone (264.271(a)); (2) demonstrate, prior to application of waste, that hazardous constituents in the waste can be completely degraded, transformed or immobilized in the treatment zone (264.272(a)); (3) design, construct, operate and maintain the unit to maximize the degradation, transformation, and immobilization of hazardous constituents in the treatment zone, to minimize run-off and run-on, and to control wind dispersal and to provide a run-off, run-on, and wind dispersal inspection program (264.273(a)-(g)); (4) establish an unsaturated zone monitoring program (264.278), and (5) implement certain operational, monitoring and inspection requirements during the closure and post-closure care periods (264.280).

As stated above, 40 CFR 264.278 requires that all land treatment units have an unsaturated zone monitoring program. This program must provide the capability of determining whether hazardous constituents have migrated below the treatment zone. The monitoring program must include the use of both soil cores and devices to measure soil-pore liquid (such as lysimeters). The unsaturated zone monitoring program system must consist of a sufficient number of monitoring points at appropriate locations and depths to yield samples that:

(1) Represent the quality of background soil-pore liquid and the chemical make-up of soil that has not been affected by leakage from the treatment zone; and

(2) Indicate the quality of soil-pore liquid and the chemical make-up of the soil below the treatment zone.

40 CFR 264.278(d) requires the owner or operator to conduct soil monitoring and soil-pore liquid monitoring immediately below the treatment zone at frequency and timing specified by the EPA Regional Administrator. The owner or operator must determine whether there is a statistically significant change over background values for all hazardous constituents required under Section 264.278 (a). This determination must be made below the treatment zone each time the owner/operator conducts soil monitoring and soil-pore liquid monitoring (Section 264.278(f)).

RCRA Section 3004(o)(4) requires a leak detection system capable of detecting leakage at the earliest practicable time for all new and existing land treatment units. This requirement will become effective 24 months after promulgation of the final liner/leak detection rule.

To satisfy the statuary leak detection requirements of RCRA, EPA will propose the following additions to the current 40 CFR 264.278 unsaturated zone monitoring requirements: (1) a 95-percent confidence level for detecting hazardous constituents below the treatment zone; (2) detection of leakage at the earliest practicable time; (3) monitoring to be conducted above the seasonal high water table; (4) a response action plan for major and widespread leakage; and (5) inspection of unsaturated zone monitoring equipment.

### 5.3.2 Approach to Statutory Requirements

The land treatment process is fundamentally different than land disposal. The land treatment process involves biodegradation of wastes in the upper layers of the soil thereby reducing the levels of hazardous constituents during the degradation process (USEPA, 1983). Land disposal essentially involves the use of "liquids management and containment" technologies. Since the land treatment and land disposal processes require fundamentally different types of waste management structures, fundamentally different approaches are required to satisfy the statutory leak detection requirement of RCRA.

EPA has elected to employ the existing unsaturated zone monitoring requirements under 40 CFR 264 to satisfy statutory leak detection requirements at new and existing permitted units. The EPA is planning to expand on the monitoring requirement by adding a monitoring confidence level of detection of 95 percent and requiring detection in the earliest practicable time. The unsaturated zone monitoring is the vehicle that EPA is proposing to be used for detection of leakage.

If the owner or operator detects concentrations of constituents statistically exceeding background levels, then appropriate operational controls will be required such as reducing the application rate.

The owner or operator of new and existing units at permitted or interim status facilities will be required to include in the permit application or operating plan a response action plan for widespread leakage. This will be prepared and submitted before waste can be received at a new unit. Leakage less than widespread will not be required to have a RAP because the treatment requirements, as noted above, have a process to address smaller leakage rates through operational changes at the unit.

### 5.3.3 Detection Confidence Level

EPA is proposing to add a 95 percent confidence level for detection of hazardous constituent migration out of the land treatment unit to the existing 40 CFR 264.278 requirements. Land treatment relies on the degradation, transformation or immobilization of hazardous waste within the treatment zone. However, land treatment units have no barrier to downward migration, and ground water can be located as close as one meter to the bottom of the treatment zone. For these reasons, EPA takes the position that the owner or operator must detect leakage out of the unit at the earliest practicable time and at a 95-percent confidence level.

By requiring a 95-percent confidence level, EPA will be assuring that the unsaturated zone monitoring system will consist of a sufficient number of sampling points at appropriate locations and depths to determine the spatial and temporal variations in constituent concentrations beneath the entire treatment zone. A properly designed and well managed site with uniform waste application will require fewer sample locations than a poorly managed site. The owner or operator must consider site-specific variations and the relative uncertainty associated with soil-pore liquid sampling procedures in developing the unsaturated zone monitoring program.

EPA is proposing the confidence level value of 95 percent as a result of recently developed guidance [USEPA, 1986a; USEPA, 1986b] on unsaturated zone monitoring. This guidance explains what is required of owners or operators to comply with the confidence level requirements.

A confidence level is the range within which the true value of a parameter is to be found with a given probability. The reliability expressed by the confidence level states the level of precision of the sampling study. Three levels of confidence are commonly used: (i) 68 percent; (ii) 95 percent; and (iii) 99 percent. These can be expressed as ± 1 standard deviation, ± 1.96 standard deviations, and ± 2.58 standard deviations of the mean, which covers 68 percent, 95 percent and 99 percent respectively. Another way to describe a confidence level is to say that the probability is 0.32 (or 1 in 3) that the value is outside of 1 standard deviation on either side of the mean, 0.05 (or 1 in 20) that the value is outside 1.96 standard deviations on either side of the mean, or 0.01 (or 1 in 100) that the value is outside 2.58 standard deviations on either side of the mean.

When results have to be absolute, a 99 percent confidence level is used. When funding or other resources are limited, or when reliability is comparatively unimportant, the 68 percent confidence level may be acceptable. Environmental sampling, however, traditionally attempts to attain a level of 95 percent confidence.

Establishing the level of confidence is often a judgment exercised by the researcher based on the degree of reliability desired. Selection of the confidence level affects many decisions relating to the research studies, including the number of samples that must be taken. Consequently, statistical sampling studies require a known confidence level to be established before sampling begins.

A 95 percent confidence level was selected by EPA because it is the generally accepted level of confidence for environmental studies, it provides a high level of reliability and it sets a reasonable standard for reliability and precision.

### 5.3.4 Monitoring Periods

The mandate of RCRA 3004(0)(4)(B) requires an approved leak detection system to be utilized which is capable of detecting leaks at the earliest practicable time. To meet this requirement, EPA is proposing to require quarterly monitoring to detect hazardous constituents at the earliest practicable time.

Two kinds of monitoring are used at land treatment units: soil core and soil-pore liquid monitoring. Soil-core monitoring is used to evaluate the transport of relatively slow-moving waste constituents while soil-pore liquid monitoring is designed to detect rapid pulses of mobile waste constituents that may be rapidly transported through the unsaturated zone to ground water and that are unlikely to be observed through regularly scheduled analysis of soil cores.

Appropriate timing of soil-pore liquid monitoring is essential, and ideally sampling is performed after precipitation or snow melt is sufficient to generate leachate (although sampling depends on specific soil and site conditions that determine the rapidity of water movement in the soil).

In the EPA's technical document on land treatment [USEPA, 1983c], quarterly sampling of soil-pore liquid is recommended although more frequent sampling may be recommended in areas of high rainfall or highly permeable soils or at land treatment sites where waste is frequently applied.

The requirement for quarterly monitoring of soil-pore liquids is considered to be a reasonable minimum frequency because even if fast-moving hazardous constituents are to move out of a treatment zone, they usually migrate within 90 days following waste application.

Soil-core monitoring is used to evaluate the transport of waste constituents which may move through the soil profile fairly slowly. Waste constituents transport may be slow because of insufficient soil moisture to leach through the system, or a natural or artificially occurring layer or horizon of low hydraulic conductivity or waste constituents that only exhibit a low to moderate mobility relative to water in soil.

EPA recommends [USEPA, 1983c] that soil-core sampling be conducted at least semi annually. Quarterly sampling therefore is a way to provide the earliest practicable leak detection for both soil-core sampling and soil-pore sampling.

### 5.3.5 Inspection

EPA is also proposing to extend current 40 CFR 264.284 regulations to require owners or operators to establish an inspection program for the unsaturated zone monitoring equipment for the active life and post-closure care period of a facility. This program will require the owner or operator to establish a mechanism for determining deterioration, malfunction, or improper operation of unsaturated zone monitoring equipment. The program also will determine the effectiveness of controls implemented in response to hazardous constituent migration beyond the treatment zone that statistically exceed background levels. The owner or operator will be required to keep a detailed log of all inspection information to demonstrate compliance with unsaturated zone monitoring permit requirements. The Regional Administrator (RA) may require additional inspection and monitoring requirements which will be specified in the permit.

### 5.3.6 Response Action Plan

Existing regulations (40 CFR 264.278) require the owner or operator to notify the Regional Administrator within 7 days when there is a statistically significantly increase in hazardous constituents below the monitoring zone. The owner or operator also must submit to the RA within 90 days an application for a permit modification to change the operating practices at the facility to maximize the success of degradation, transformation, or immobilization processes in the treatment zone.

EPA is proposing a requirement that after the effective date of the liner/leak detection rule, owners or operators of land treatment units develop response action plans (RAPs) for "widespread leakage" before waste is received. The RAP will set forth actions to be taken upon finding widespread leakage. Widespread leakage is defined as a statistically significant increase of hazardous constituents at 50-90 percent or more of the unsaturated zone sampling points. This increase represents migration from the total areal extent of the unit posing a threat to groundwater quality. Widespread leakage requires an immediate response action plan.

The EPA believes that isolated leakage (a statistically significant increase of hazardous constituents at fewer than 50 percent of the unsaturated zone sampling points) presents a minimal threat to ground water design and operation may need to be adjusted within the unit to minimize local hazardous constituent migration out of the land treatment unit. However, if statistically significant increases in hazardous constituents occur at 50 to 90 percent of the unsaturated zone sampling points (i.e., widespread leakage), then it is believed the threat to human health and the environment at this level of migration out of the unit increases to the point that substantive changes in the operation of the unit or closing the unit may be necessary.

The possible courses of action to take upon finding widespread leakage include changing the operating practices or closing the facility. Changing operating practices may include changing the type of waste treated, the timing of application, a reduction of the amount applied, or a reduction in the application frequency. Closing the facility may be necessary if changing operating practices cannot be shown to protect ground and surface water or if the owner or operator finds the changes to be cost prohibitive.

EPA considered other possible response actions for widespread leakage, but chose not to include them. These actions include increasing the frequency of groundwater monitoring, excavating the unit, and installing a cover over the unit. The EPA has taken the position that using increased groundwater monitoring would be too slow to detect contamination and does not achieve the goal of preventing groundwater contamination. Excavating a land treatment unit and disposing of the contaminated soil would be excessively expensive and would achieve protection similar to closing the unit. The last option considered is installing a temporary landfill cover over the unit or part of the unit. This is counter to the principles of land treatment, which are to allow natural aerobic processes to degrade waste.

## 5.4 INTERIM STATUS LAND TREATMENT UNITS

### 5.4.1 Current Regulations

EPA's current regulations (40 CFR 265, Subpart M) require owners or operators of interim status land treatment units to install and operate an unsaturated zone monitoring system with the capability of detecting the migration of hazardous constituents vertically from the land treatment facility's treatment zone.

Data on the background concentrations of hazardous waste and hazardous waste constituents also must be provided by the monitoring plan, which must include the use of soil cores for soil monitoring and lysimeters (or a similar device) for soil-pore liquid monitoring.

The current regulations for interim status land treatment unit unsaturated zone monitoring are less stringent than those proposed by EPA. Current regulations are:

(1) The owner and operator must demonstrate that the depth at which soil and soil-pore water samples are taken is below the treatment zone;

(2) The owner and operator must base the number of soil and soil-pore water samples on the variability of the hazardous waste constituents and soil types. Furthermore, sampling frequency and sampling time must be based on the frequency, time, and rate of waste application, proximity to ground water, and soil permeability.

A copy of the unsaturated zone monitoring plan and the rationale used to develop the plan must be filed at the land treatment facility by the owner or operator according to 40 CFR 265.278 (d).

The owner and operator also must analyze the soil and soil-pore water samples for hazardous waste constituents that exhibit an extraction procedure (EP) toxicity characteristic (40 CFR 265.273(a)), and he must determine the presence and concentration in the sample of any substances on the federal hazardous waste list specified in CFR 261, Subpart D (40 CFR 265.273(b)).

The Hazardous and Solid Waste Amendments of 1984 (RCRA 3004(o)(4)) require interim status land treatment facilities to utilize an approved leak detection system at the earliest practicable time. This requirement becomes effective 24 months after the final liner/leak detection rule is promulgated.

To satisfy the statutory requirements, EPA is proposing to require interim status units to achieve the same monitoring standards as permitted units. The more stringent standards are based on the proposition that interim status land treatment poses the same threat of adverse impacts to human health and the environment as permitted land treatment. Therefore, EPA believes existing units can comply with the standard in the same manner as new units and thus provide the same level of protection to human health and the environment.

### 5.4.2 Proposed Interim Status Monitoring Requirements

EPA proposes to replace current regulations governing interim status land treatment monitoring (40 CFR 265.278) with present and proposed additional monitoring requirements (40 CFR 264.278) for permitted land treatment units. These proposed regulations will require essentially the same monitoring plan at all land treatment units. The upgraded monitoring plan that is proposed has the following additional elements: (1) a 95 percent confidence level for detecting hazardous constituent migration from the treatment zone, (2) detection of hazardous constituent leakage at the earliest practicable time, (3) conduction of soil and soil-pore liquid monitoring above the seasonal high water table (SHWT), (4) a response action plan for widespread leakage and (5) unsaturated zone monitoring equipment inspections.

### 5.4.3 Interim Status Implementation Differences

Proposed requirements for permitted and interim status land treatment units are similar, but implementation procedures differ in two ways: (1) The interim status land treatment unit owner or operator must have a written leak detection plan at the facility and (2) a copy of the plan must be sent to the EPA Regional Administrator.

The minimum requirements for a written interim status leak detection plan are:

> (1) A description of how soil and soil-pore liquids will be monitored to determine at the earliest practicable time whether hazardous constituents have migrated out of the treatment zone. This monitoring program must cover all areas likely to be exposed to hazardous waste and leachate through the facility's active life and post-closure care period. Hazardous constituents or principal hazardous constituents to be monitored must be identified.
>
> (2) A description of the number, location and depth of soil-pore liquid monitoring devices necessary to represent to a 95 percent confidence level the following characteristics;

a) soil and soil-pore liquid quality below the treatment zone,
b) the quality of background soil and soil-pore liquid,

(3) A description of methodology for establishing background values for each hazardous constituent to be monitored.

(4) A description of the frequency, time, and depth of soil and soil-pore liquid monitoring based on the frequency, time and rate of waste application and on the soil permeability.

(5) A description of sampling and analysis procedures that are designed to ensure that sampling results provide a reliable indication of soil-pore liquid quality and the chemical composition of soil below the treatment zone. Procedures for sample collection, preservation and shipment, along with analytical procedures and chain of custody control should be included.

(6) A description of the statistical procedure to determine if there is a significant increase over background values in the monitoring data. This description must include the amount of time allowed between drawing a sample and determining the statistical significance of that sample. The plan must specify a statistical procedure that is appropriate for the distribution of data used to establish background values. This procedure must provide a reasonable balance between the probability of a false determination and failure to identify migration of hazardous waste.

(7) A response action plan that describes action to take if it is determined that a widespread leakage has occurred.

### 5.4.4 Monitoring Plan Amendments

Once a statistically significant increase of hazardous waste constituents is determined by the owner or operator to have occurred at an interim status facility, the EPA Regional Administrator must be contacted in writing within 7 days. This notice must identify the constituents detected and include the preliminary concentration levels. An amendment to the operating plan must be submitted in 90 days of the determination. This plan must show that operating practices have been modified sufficiently to maximize the success of degradation, transformation, or immobilization processes in the treatment area.

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